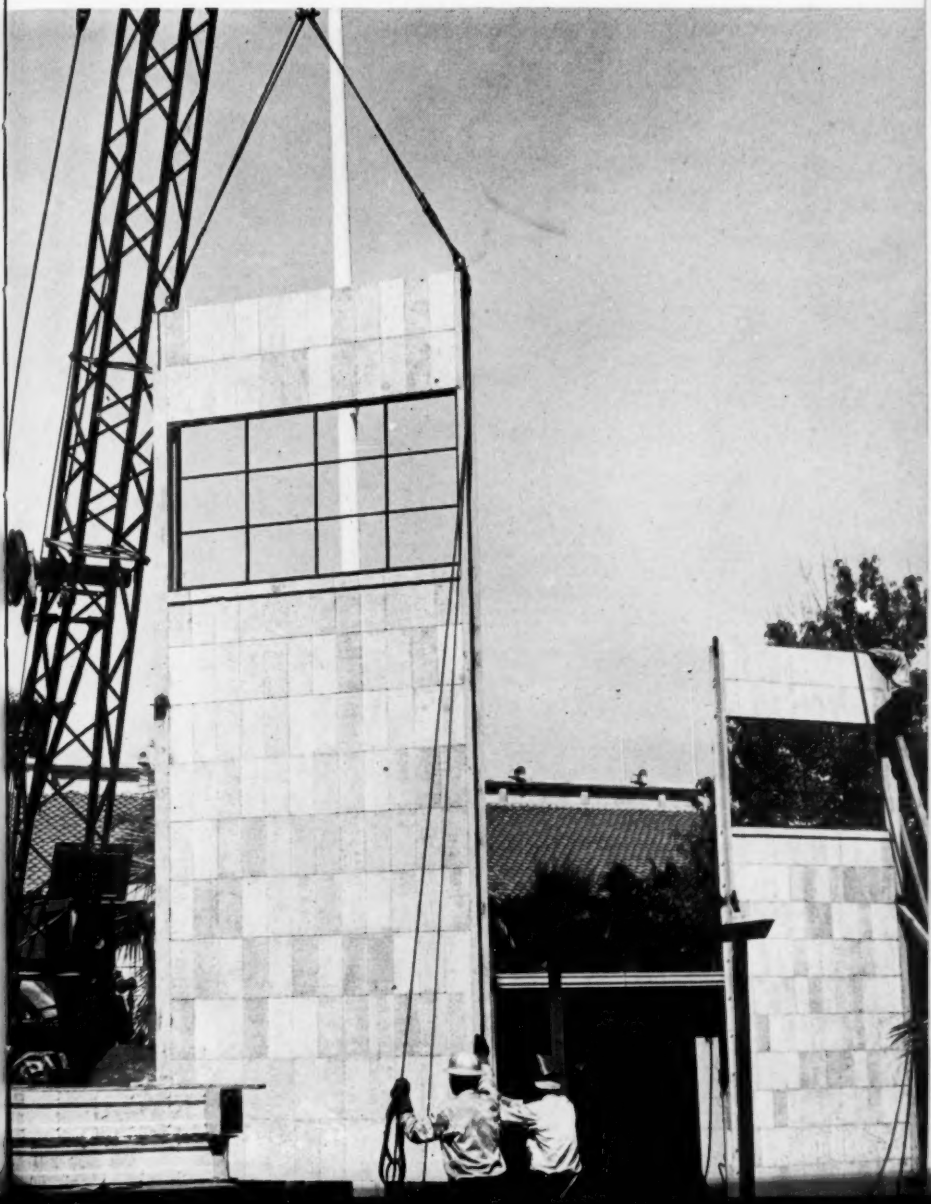
ACI News Letter

Vol. 23 No. 2 JOURNAL of the AMERICAN CONCRETE INSTITUTE October 1951



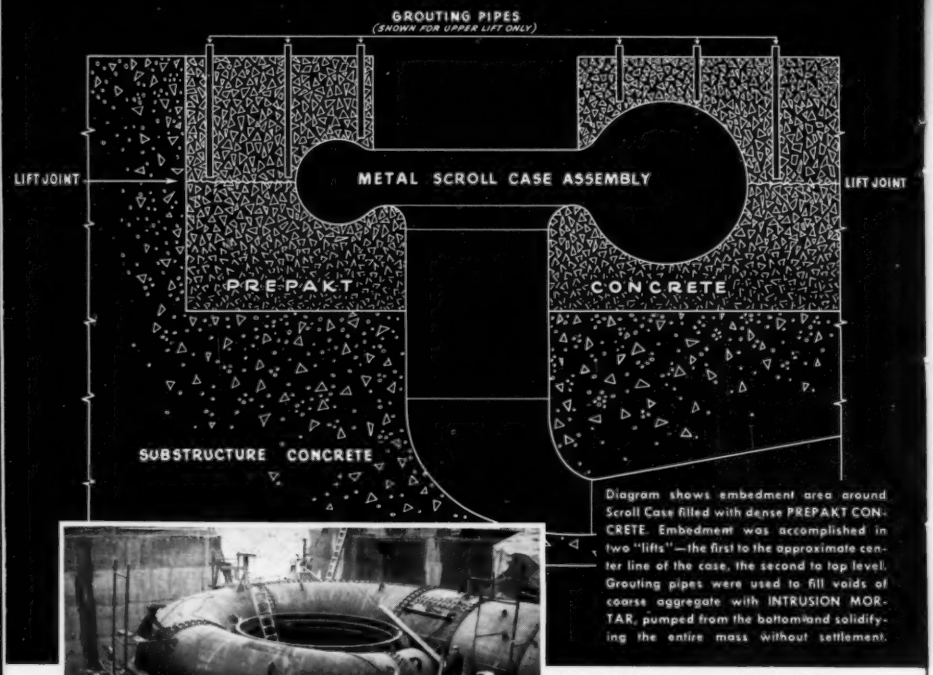
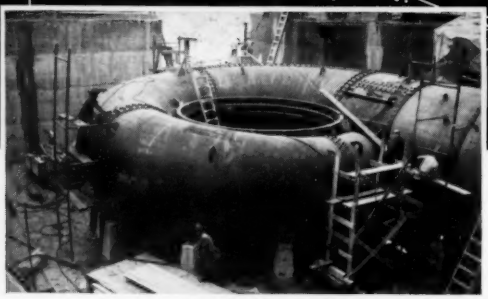


Diagram shows embedment area around Scroll Case filled with dense PREPAKT CONCRETE. Embedment was accomplished in two "lifts"—the first to the approximate center line of the case, the second to top level. Grouting pipes were used to fill voids of coarse aggregate with INTRUSION MORTAR, pumped from the bottom and solidifying the entire mass without settlement.



View of Turbine Scroll Case before embedment with PREPAKT CONCRETE, showing conventional concrete substructure. The case was embedded later to a level even with the top of the substructure.

PREPAKT *makes a* BIG DIFFERENCE IN SETTING TURBINE SCROLL CASES

Not one, but several major advantages are found in the application of PREPAKT methods and materials for embedding Turbine Scroll Cases.

First, the PREPAKT encasement is accomplished with two "lifts" instead of the eight, ten, or more required with conventional methods. Then too, Intrusion materials prevent setting and settling shrinkage, assuring complete bond and full underpinning strength. The entire procedure is accomplished without strain or distortion to the Scroll Case in approximately 50% of the time required under standard concreting methods.

Full information on this or other PREPAKT applications may be had by writing Prepakt Concrete Co., Cleveland, Ohio.

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ACI

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October 1951

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Goldbeck receives nomination for president

Scholer nominated for vice-president

The 1951 ACI Nominating Committee, consisting of Raymond E. Davis, chairman, Robert F. Blanks, Waldo G. Bowman, Roy W. Carlson, H. J. Gilkey, F. H. Jackson, T. E. Stanton and Charles S. Whitney, reports the following nominations for election at the 48th annual convention in Cincinnati:

For President

A. T. Goldbeck to succeed Harry F. Thomson for a one-year term beginning at the February 1952 convention.

Mr. Goldbeck, engineering director of the National Crushed Stone Assn., Washington, D. C., has been active in ACI affairs for many years. He served on the Advisory Committee, 1922-23; Publications Committee, 1944-46; Board of Direction, 1948-51. He is a member of Committee 115, Research; Committee 325, Structural Design of Concrete Pavements for Highways and Airports; Committee 613, Recommended Practice for the Design of Concrete Mixes; Committee 617, Specifications and Recommended Practice for Concrete Pavements and Bases; and Committee 621, Aggregate Selection, Preparation, Handling and Use. He has been chairman of ASTM Committee C-9 on Concrete and Concrete Aggregates, chairman of the Design Committee of the Highway Research Board and member of two of the Joint Committees on Concrete and Reinforced Concrete.

Mr. Goldbeck graduated from the University of Pennsylvania with a BS degree in civil engineering in 1906, and served as instructor at the University for two years.

Cont'd on p. 4

He joined the staff of the office of Public Roads in 1910, became assistant engineer in charge of the Philadelphia Municipal Laboratory in 1913, rejoined the reorganized laboratory of the Bureau of Public Roads and Rural Engineering in 1915, and served successively as engineer of tests and chief of the Division of Tests. In these capacities he conducted and directed research in concrete pavement design which included studies of stress distribution under static and moving loads, subgrade pressure measurements, studies of subgrade friction, impact studies, the flow of concrete, measurements of expansion and contraction, and other studies pertaining to pavement design.

In January 1951, Mr. Goldbeck received the Distinguished Service Award from the Highway Research Board for "outstanding achievement in the field of highway research," and in June 1951 he became an Honorary Member of ASTM. Early in his career, he was honored by election to membership in Sigma Xi.

For Vice-President

C. H. Scholer for 2-year term, beginning at the February 1952 convention.

Professor Scholer, Head, Department of Applied Mechanics, Kansas State College, Manhattan, Kan., has been an ACI member since 1924. He has been active on Committee 115, Research, and Committee 215, Fatigue of Concrete.

After receiving a BS degree in Civil Engineering from Kansas State College in 1914, he was head chainman, U. S. General Land Office surveys, Santa Fe, N. M., 1914-15; chainman, maintenance of way and realignment problems, Atchison, Topeka and Santa Fe R. R., Topeka, Kan., 1915-16; assistant engineer in charge of surveys and preparation of plans, and supervision of the construction of roads, pavements and bridges for the Extension Division, Kansas State College, Manhattan, Kan., 1916-17; assistant bridge engineer in charge of the design and supervision of construction of highway bridges for the Kansas State Highway Commission, 1917-18; professor and head, Department of Applied Mechanics, since 1919 except for a leave of absence to take charge of investigational work in New York and Pennyslvania for the Portland Cement Assn., February 1939 to September 1940.

Professor Scholer is associated with ASCE, NSPE, ASTM, ASEE, Kansas Engineering Society, the Highway Research Board and has been honored by election to Sigma Xi and Phi Kappa Phi.

Henry L. Kennedy, manager of the Cement Division, Dewey and Almy Chemical Co., Cambridge, Mass., elected for a 2-year term as vice-president at the 1951 convention in San Francisco, continues in that office.

Directors

For Directors, 3-year terms, four nominations for as many places on the Board: A. A. Bates, J. W. Kelly, F. N. Menefee and Bailey Tremper.

Dr. Bates, vice-president of research and development, PCA, Chicago, holds degrees from Ohio Wesleyan University and Case Institute of Technology and a D. Sc. *magna cum laude* from the University of Nancy in France. He has an honorary degree of Doctor of Engineering from Stevens Institute of Technology and a PhD from Rose Polytechnic Institute. Dr. Bates was professor of metallurgical engineering at Case Institute of Technology for ten years and was manager of the chemical, metallurgical and ceramic research division of the Westinghouse Electric Corp. before joining PCA in 1946.

Dr. Bates is a member of Sigma Xi and Tau Beta Pi.

Professor Kelly, Civil Engineering Department, University of California, Berkeley, Calif., has been an ACI member since 1926 and is chairman of Committee 611, Inspection of Concrete. He was graduated from Purdue University in 1921. For two years he was engaged in waterworks engineering, making valuations and superintending construction. After a brief term as assistant in the testing materials laboratory of Purdue University, he joined the staff of PCA where he remained for seven years. In May 1931, he joined the faculty of the University of California.

Professor Menefee, professor of engineering materials, University of Michigan, Ann Arbor, Mich., has been an ACI member since 1925 and is chairman of Committee 711, Precast Floor Systems for Houses. He received his BS degree in civil engineering from the University of Nebraska in 1908 and CE degree in 1932. In 1910 he received a CE degree from Cornell and in 1937 a

doctor's degree in civil engineering from Lawrence Institute of Technology, Detroit, Mich. Since 1919 he has been Professor of Civil Engineering, University of Michigan, combining his teaching work with many practical applications.

He is a member of ASCE, ASTM, Kappa Sigma, Sigma Xi, Sigma Rho Tau and an honorary member of Sigma Tau.

Mr. Tremper, materials and research engineer, Washington State Highway Dept., Olympia, Wash., is a member of Committee 115, Research, and Committee 617, Specifications and Recommended Practice for Concrete Pavements and Bases. He was graduated from the University of Washington, Seattle, in chemical engineering in 1914 and became assistant chemist for the Superior Portland Cement Co., Inc., Concrete, Wash., where he served until his enlistment in World War I in the Chemical Warfare Service. Upon return to civilian life, he became chief chemist for the Superior Portland Cement Co., and in 1921 he joined the staff of the Washington Department of Highways.

Nominating Committee

The 20 candidates for membership on the 1952 Nominating Committee—five to be elected—are:

F. H. Bauman	N. M. Newmark
S. J. Chamberlin	J. R. Nichols
H. F. Clemmer	D. E. Parsons
H. K. Cook	R. W. Spencer
H. E. Davis	M. A. Swayze
H. C. Delzell	L. H. Tuthill
Fred Hubbard	G. W. Washa
J. E. Jellick	C. A. Willson
William Lerch	K. B. Woods
C. A. Menzel	D. O. Woolf

Larson named head of USBR region

Vaud E. Larson, for the past five years assistant regional planning engineer for the U. S. Bureau of Reclamation at Phoenix, Ariz., has been named head of the Bureau's Region 3, Branch of Project Planning, at Boulder City, Nev. Mr. Larson will supervise investigations for potential irrigation and reclamation developments in southern California and Nevada and parts of Arizona, Utah and New Mexico.

United States prestress conference

The first United States conference on prestressed concrete held at the Massachusetts Institute of Technology, Cambridge, August 14-16, 1951, was a great success, having a registration of over 600 representing every state in the United States as well as Canada, England, France, Portugal, Cuba and Venezuela.

During this conference sponsored by MIT, ACI, AGC, AIA, ASCE, AREA and PCA, it was suggested by several speakers that the advisory committee should be kept active. This is going to be done under the chairmanship of Prof. Myle J. Holley.

The following resolution by Frederick S. Merritt of *Engineering News-Record* was adopted unanimously at the final session:

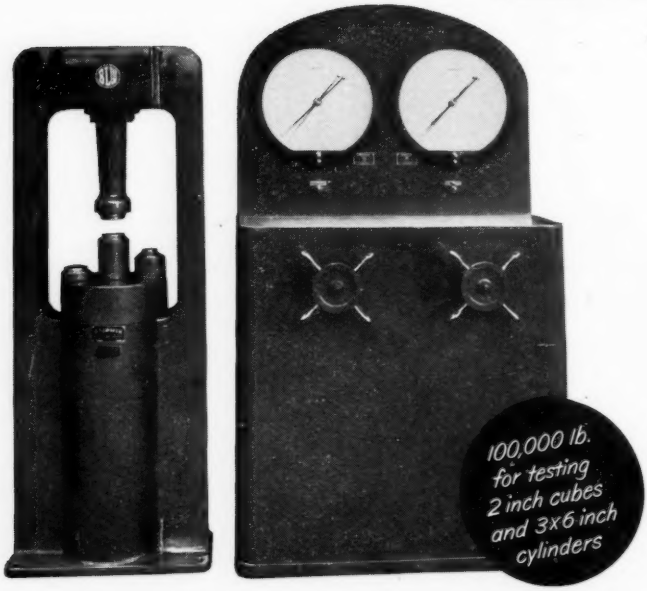
"It is the consensus of the gathering at the first U. S. conference on prestressed concrete held at the Massachusetts Institute of Technology, August 14-16, 1951, that it is now necessary to crystallize nomenclature and design criteria for prestressed concrete to avoid delay in acceptance of practical applications in construction, and it is therefore urged that the American Concrete Institute prepare as soon as possible a proposed standard for nomenclature and design in such general form that it will not restrict the development of various known and future methods and their applications to industry and construction."

Awards for annual reports

Five thousand corporation annual reports for 1950 were rated in the 11th annual survey of *Financial World* with seven cement companies, all members of ACI, qualifying for "Highest Merit" citations. Winners were Bessemer Limestone and Cement Co., Consolidated Cement Corp., General Portland Cement Co., Lehigh Portland Cement Co., Lone Star Cement Corp., Marquette Cement Manufacturing Co., Missouri Portland Cement Co. and Permanente Cement Co.

The stockholder reports of these companies thus become candidates for the final judging, and one will be selected Oct. 29, 1951 for a "Best of Industry". award. A year ago the 1949 annual report of Marquette Cement Manufacturing Co. won the trophy for the best report in this industrial classification.

NOW...TWO BALDWIN sized for the



Recently redesigned to separate the loading and weighing units, this modern 100,000 lb. Baldwin Concrete Tester offers new precision and safety in operation, new attractiveness in appearance.

The two-unit design prevents transmission of load shocks to the indicator, and also keeps the operator out of range of flying particles from breaking specimens. Clear space is 17⅜ inches between columns. Maximum opening between ram and upper platen is 22¼ inch. Ram travel is three inches, at speeds up to 1¼ inch per minute.

Ranges on two dials are 0-100,000 and 0-10,000 lbs., graduated in 200 and 20 lb. increments, respectively. Accuracy is within ±¾% of indicated load, or 0.15% of full scale capacity, whichever is greater.

This machine not only handles routine testing of 2 inch cubes and 3x6 inch cylinders, but is suitable for many other uses. Ask for Cat. 327.

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Especially suited to rapid, precise testing of standard concrete test specimens 3 x 6 inches, 6 x 12 inches and 8 x 16 inches, this 300,000 lb. Baldwin Concrete Tester can, by the addition of special bearing blocks, be immediately adapted for testing 12 x 18 inch building blocks.

Exposed piston surface is protected from dust and wear by bellows-action canvas cover. High range dial has 500 lb. divisions up to 300,000 lbs. Low range scale runs to 50,000 lbs. in 100 lb. divisions. Loading and indicating units are separate, to eliminate transmission of shock and eliminate operator hazard from flying or falling debris. For full details, ask for Bulletin 318.

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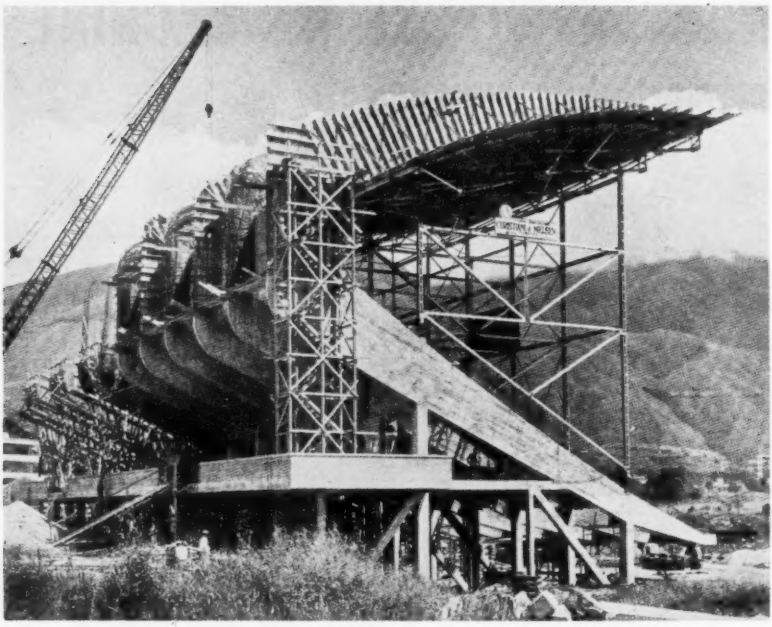


Photo courtesy Danish Foreign Office Journal

Stadium at Caracas, Venezuela, built by Christiani & Nielsen

Christiani & Nielsen

The firm of Christiani & Nielsen, active in engineering work in many parts of the world, was founded in Copenhagen in 1904 to design and build reinforced concrete structures. It established its first foreign branch in Hamburg in 1908 probably because this large port offered unusual possibilities for the firm's specialized quay construction methods. A series of well conceived and executed reinforced concrete projects established the firm as one of the pioneers in modern harbor building techniques.

In 1910 a branch was opened in Russia, then undergoing rapid technical and economic development. The branch grew quickly and was entrusted with many important undertakings, but was stopped by the revolution of 1917.

Note: This is the first of what it is hoped will become a series of thumb-nail sketches (to appear at irregular intervals) of well-known ACI Corporation members of 20 years or more continuous membership.—Editor

In 1913 a branch office was opened in England, following the award of a contract for pier work for the Tunnel Cement Factory. The branch undertook all forms of civil construction and gradually gained a recognized position. In 1916 the firm established an office in Norway, and in 1917 one in Sweden.

The anticipated increase in industrial and other activity all over the world, following the Armistice of 1918, led to the establishment of a further series of foreign branches. In 1919, an affiliated company was formed in France which was relatively unimportant until in 1928 it was awarded some big repair jobs under the Dawes Plan. One of the first of these was the transatlantic railway terminal at Cherbourg, with adjoining quays, and the dock gates at St. Nazaire, designed also as a dry dock for the Normandie.

In 1918 Christiani-Nielsen Engenheiros E Constructores, SA was opened in Brazil and became not only the firm's biggest overseas

branch but also the biggest engineering firm in Brazil. A branch in Argentina, opened in 1919, soon became one of the leading firms in that country.

In 1930 Christiani & Nielsen, in association with the East Asiatic Co., established an affiliate in Siam. More recently the firm has undertaken various projects in Australia and New Zealand.

With 24 branches and affiliates in 24 countries, Christiani & Nielsen is presently working on about 300 contracts in all parts of the world.

Bridge tests completed

Composite action of steel and concrete is definitely shown in the results of tests on the San Leandro Creek Bridge, south of Oakland, Calif. The bridge has been the subject of an extensive series of tests aimed at determining actual stresses in the structure.

Strain gages at more than 200 points on the bridge structure were utilized in taking measurements under both static and dynamic loadings. Dynamic loads were provided by 70,000- and 85,000-lb. vehicles, run over different parts of the bridge at speeds varying from 3 to 25 mph. Some 20,000 ft of oscillographic strain recordings were obtained.

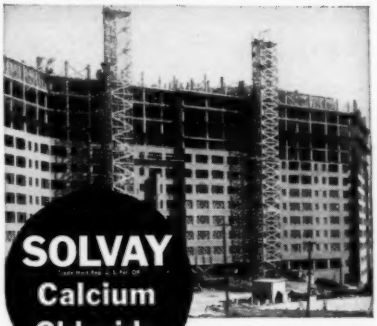
Field testing was virtually completed in June. While it may take months to complete the reduction of data and analyses, an interim report will be issued this fall. Analysis to date, however, has definitely indicated the composite action of the concrete deck and the steel girders. This holds forth the possibility of designs which, by taking composite action into account, might result in considerable savings of steel.

The tests were a cooperative project of the Institute of Transportation and Traffic Engineering of the University of California, the U. S. Bureau of Public Roads, and the California Division of Highways. In addition, the U. S. Coast and Geodetic Survey took measurements to determine the natural frequency of vibration of the bridge.

Sheets addresses research group

Frank T. Sheets, president of the Portland Cement Assn., Chicago, Ill., recently addressed coal and railroad executives of Bituminous Coal Research, Inc., in Columbus, Ohio, on the subject of "Applying Research Results to Markets."

For Concrete Construction....



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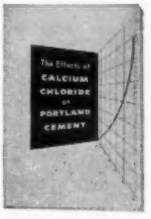

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Who's Who This Month

F. N. Menefee

F. N. Menefee, professor of engineering mechanics, University of Michigan, Ann Arbor, Mich., describes "Advances in Precast Floor Systems," p. 113.

Professor Menefee has been an ACI member since 1925 and is chairman of Committee 711, Precast Floor Systems for Houses. He has a BS degree in civil engineering from the University of Nebraska and a doctor's degree in civil engineering from Lawrence Institute of Technology, Detroit, Mich. Since 1919, he has been Professor in charge of Civil Engineering, University of Michigan. Presently, he is teaching a graduate course in prestressed reinforced concrete.

Professor Menefee is a member of ASCE, ASTM, Kappa Sigma, Sigma Xi, Sigma Rho Tau and is an honorary member of Sigma Tau. He has just been nominated for Director of ACI (See p. 4).

A. G. Streblow

A. G. Streblow, president, Basalt Rock Co., Inc., Napa, Calif., is the author of "Manufacture and Use of Machine-Made Precast Structural Elements," p. 125.

Mr. Streblow is vice-president and director of the National Concrete Masonry Assn. and serves on the Technical Problems Committee of that association.

In 1937, he went to Europe to study methods of manufacturing lightweight concrete units and precast slabs.

Mr. Streblow has been an ACI member since 1946.

F. Thomas Collins

F. Thomas Collins, consulting engineer, San Gabriel, Calif., presents "Tilt-Up Construction in Western United States," p. 133.

After receiving BA and ME degrees from Stanford University in 1934, Mr. Collins was employed by the Union Oil Co. at Santa Fe Springs, Calif., followed by four years with Douglas Aircraft as a strength test engineer.

He is secretary-treasurer of the Precast Concrete Assn., Ltd., a newly organized association dedicated to furthering the advancement of prestressed and precast concrete.

Mr. Collins is a recognized authority on precast concrete design and construction and is now teaching the subject.

He has been an ACI member since 1949.

Arthur P. Clark

Arthur P. Clark, research associate, American Iron and Steel Institute, National Bureau of Standards, Washington, D. C., contributes "Diagonal Tension in Reinforced Concrete Beams," p. 145.

After graduating from the University of Michigan in 1903 with a BS in civil engineering, he spent 3½ years in the bridge department of the Pere Marquette Railroad. He has been associated with reinforced concrete engineering and reinforcing distribution studies since 1906, first with the Corrugated Bar Co. of St. Louis and then continued with the company when it consolidated with Kalman Steel Co. and was later purchased by Bethlehem Steel Co. He has been at the Bureau of Standards since 1944.

Author of many JOURNAL papers, Mr. Clark has been a member of ACI since 1924 and is a member of Committee 208, Bond Stress, and Committee 326, Shear and Diagonal Tension (an ASCE-ACI joint committee). He is also a member of ASCE, ASTM and the Highway Research Board.

J. G. Dempsey

J. G. Dempsey, chief engineer, Sollitt Overseas Construction Co., San Juan, Puerto Rico, writes about "Coral and Salt Water as Concrete Materials," p. 157.

Mr. Dempsey is interested in studies of the durability of concrete in service as well as concrete control and inspection and is presently directing concrete operations and field engineering during construction of Loiza Dam in Trujillo Alto, P. R., a gravity structure containing about 70,000 cu yd of concrete. He is also field consultant on concrete problems for the Puerto Rico Water Resources Authority relative to the Yauco

Dam, a hydroelectric and water-supply project.

Receiving his technical training at the University of Vermont and Massachusetts Institute of Technology, Mr. Dempsey was employed by the highway departments in Vermont and Missouri on highway construction work. In 1934 he was employed by the U. S. Engineer Department on the New England flood control program on the design and construction of concrete flood works. Under the same department, he was also assigned to the construction of Tuscaloosa Lock and Dam at Tuscaloosa, Ala., as concrete and layout inspector. In 1941 he was employed by the F. H. McGraw Co. as concrete engineer on the construction of the U. S. Naval Operating Base at Bermuda.

Mr. Dempsey is a member of ACI, ASCE and the American Association for the Advancement of Science.

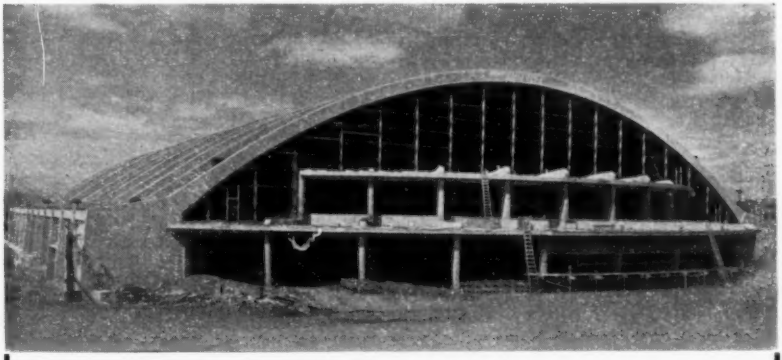
Henri Marcus

"The Bearing Capacity of Dowels as Load-Carrying Devices for Transverse Joints of Concrete Pavements," p. 169, introduces Henri Marcus, research consultant, Mechanics Division, Naval Research Laboratory, Washington, D. C.

Dr. Marcus was connected with the development of reinforced concrete in Europe for 30 years and with design and construction of a large number of bridges, sluices, water dams, harbor facilities, power plants and factories. He worked also with the well-known French engineer, Eugene Freyssinet, in the early applications of prestressed concrete. Dr. Marcus came to the United States from France in 1942 and, since that time, has worked as a consultant for the Navy Department. During World War II, he designed the first prestressed concrete barge built in this country.

He is the author of numerous publications dealing with problems related to the strength of materials and structures. His book on the theory of elastic webs and slabs and his method for the design of continuous two-way slabs are well known.

Dr. Marcus has been an ACI member since 1943.



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Next time you order the improved Bethlehem Reinforcing Bar, specify the bar by number, rather than by fractional size.

The Bethlehem Bar is now identified by numbers from #2 to #11, in accordance with Simplified Practice Recommendation R26-50 of the U. S. Dept. of Commerce. Under this new system of marking, each number denotes approximately the eighths-of-an-inch in the nominal diameter of the bar.

Bars #9, #10 and #11 are equal in weight and nominal area to the old type 1 in., 1⅛ in., and 1¼ in. square bars, respectively. With the exception of the #2 bar, which is plain round, all other sizes of the Bethlehem Bar are deformed. On each deformed bar the identifying number is branded at regular intervals to become an integral part of the deformation.

All sizes of the Bethlehem Bar are rolled as round sections. No change has been made in the standard unit weights or nominal cross-sectional areas. Therefore there is no need to change design practices, as far as they concern bar section.

The Bethlehem Reinforcing Bar is a high-quality bar. It is made from new-billet steel which meets ASTM Specification A15. It provides a firm bond in concrete because its high, uniformly spaced lugs meet the deformation requirements of ASTM Specification A305. These alternately sloped lugs also minimize slippage at working loads, and help to minimize the formation of wide tensile cracks in the concrete.

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Recently printed descriptive sheet gives full information about Bethlehem Reinforcing Bar. Keep one on hand for reference. Ask for a copy.

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Positions and Projects — ACI Members

Cook appointed ASTM subcommittee chairman

Herbert K. Cook, chief of the Concrete Research Division of the Waterways Experiment Station, Jackson, Miss., has been appointed chairman of a newly established subcommittee on methods of testing concrete for freezing and thawing. The subcommittee will be a part of ASTM Committee C-9 on Concrete and Concrete Aggregates. Mr. Cook was selected to head this new group because of his long association with the pioneer work of the Corps of Engineers in this field. In 1942 he designed the first fully automatic laboratory apparatus for testing concrete for resistance to freezing and thawing and has supervised the development and use of similar apparatus since that time.

Building Research Congress

At the Building Research Congress held in London, England, Sept. 11-20, 1951, many excellent papers on various aspects of concrete technology were presented by prominent members of the engineering profession.

D. E. Parsons presented a paper by Robert F. Blanks, USBR, "Concrete Quality Control," in which he stated that concrete quality control begins with design and must be carried through cement manufacture; aggregate exploration, selection and production; batching, mixing, handling, placing, curing and protection. Trained people must be made responsible for these operations and be given the necessary authority by top management. Modern concrete control requires screening of aggregate, expert mix design, accurate batching by weight and vibration compaction.

"Quality Control of Concrete with Particular Reference to the Economic Aspects," by C. L. a'Court and R. Ward, Imperial Chemical Industries, England, dealt with the economic savings which can be achieved by the adoption of "controlled concrete" on large-scale works using central batching plants. Reference was made to published information relating to such sites, notably the London Airport at Heathrow.

In "Curing Concrete at Elevated Temperatures," R. W. Nurse described steam curing as the most important method of obtaining high-early strength in concrete products. Methods of steam curing were discussed with special reference to the effects which rate of heating, time and temperature of curing may have on the properties of the finished concrete. High-early strength may be obtained by either low- or high-pressure steam curing, but only the latter process has an appreciable effect on the dimensional stability or on sulfate resistance.

Other papers on concrete were: "Concrete Quality Control in Building Construction," by R. L'Hermite, Director, Laboratoires du Batiment et des Travaux Publics, France; "Lightweight Concretes: A Review," by T. Whitaker, Building Research Station, England; and "The Durability of Concrete in Engineering Structures," by H. F. Gonnerman of the Portland Cement Assn.

House construction trends

The Housing Materials Use Survey of the Housing and Home Finance Agency reports interesting data on what goes into single dwellings. Concrete slab on ground was used for first floor construction in 22 percent of the houses built in 1950 whereas in 1940 this method was so little used that it was unreported. On the other hand, masonry wall construction remained static at 11 percent for both years. Coincident with the increase of slab on ground construction the number of homes with full or partial basements declined from 69 percent to 39 percent. The average area of one-story houses declined from 1009 to 941 sq ft in the ten-year period reported.

Carpenter elected president of engineering society

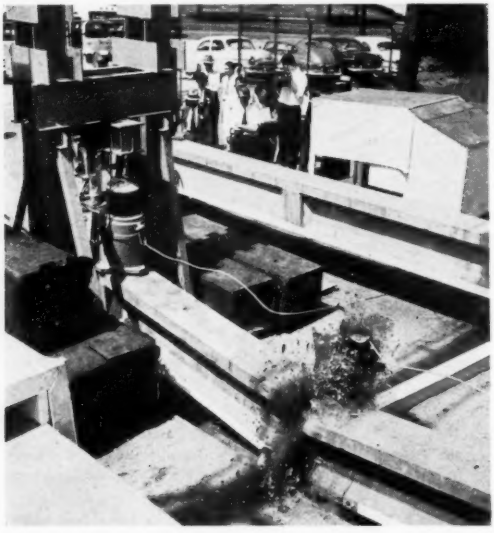
James Donald Carpenter, vice-president of Gannett, Fleming, Corddry and Carpenter, Inc., consulting and contracting engineers, Harrisburg, Pa., was elected president of the Pennsylvania Society of Professional Engineers at the organization's 16th annual convention in Allentown.

Prestressed Beams Tested

By J. R. JANNEY and THOR GERMUNDSSON

A well-planned exploration into the field of prestressed concrete was undertaken recently by The Austin Company in Cleveland, Ohio. The main purpose was to investigate problems concerning design and construction procedures to ascertain whether they could be applied with success under American conditions to a typical roof structure of a one-story industrial building with 40 x 60-ft panels.

On August 9, 1951, two beams with spans of 40 and 60 ft, respectively, were tested to destruction before a representative gathering of approximately 150 engineers. Fig. 1 shows the general arrangement of the testing area. The girder in the background was not loaded on that date. The girder in the foreground had a span of 40 ft and is shown at the moment the failure occurred. It was loaded at the quarter-points and failed near midspan after having deflected considerably for a long time. Toward the end, the tensile wires stretched plastically under a slightly increasing load until the concrete finally failed. The breaking load was approximately four times the design load. After the break occurred at midspan, cracks closed up almost completely in each half, indicating effective bonding of the wire.



Austin Company Photo

Fig. 1—Failure of 40-ft prestressed beam under test load.

Fig. 2—Sixty-ft prestressed beam just before failure.

Austin Company Photo

The 40-ft beam was 26 in. deep, and its width was 18 in. at top flange, 12 in. at bottom flange, and 5 in. in the web. It contained 48 wires of 0.192-in. diameter grouped in 4 Freyssinet cables of 12 wires each.

During the lunch hour the jacks through which the load was applied were moved from the 40-ft beam to the 60-ft beam seen in Fig. 2. The larger beam was 40 in. deep, and the width was 30 in. at top flange, 12 in. at bottom flange, and 6 in. in the web. It contained 8 Freyssinet cables with 12 wires each.

Fig. 2 illustrates the condition of the 60-ft beam shortly before failure occurred at a load of approximately four times design load. The final deflection was more than 2 ft. Load was applied at the third points and failure occurred at one of the loading points. Both girders exhibited the same deflection characteristics. Deflection in the design load range, and even up to twice design load, was unusually small and the recovery upon removal of load almost complete. When the loads approached the ultimate capacity, however, the beams kept on deflecting for a long time under increasing load without any apparent impairment of their structural integrity. When failure finally came, the break occurred suddenly in the concrete.

The load tests, which included readings of 12 electric strain gages in each beam, were performed only a few days before the First U. S. Conference on Prestressed Concrete at Massachusetts Institute of Technology. Some of the test data were included in a significant paper presented at the conference by J. K. Gannett and A. T. Waidelich, both Vice-Presidents of The Austin Company. Dr. A. R. Anderson was in charge of the installation and reading of instruments used on the test beams.

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TABLE OF ASTM A305 SPECIFICATIONS

		NOMINAL DIMENSIONS ROUND SECTIONS			REQUIREMENTS OF DEFORMATIONS		
Bar-No. *	Unit Wt. Lbs./Ft.	Diameter-Inches Decimal	Cross Sectional Area Sq. Inches	Perimeter	Max. Avg. Spacing In.	Min. Height Inches	Max. Gap. Inches °
2†	0.167	0.250	0.05	0.785	.. ..	.. .	.. ..
3	0.376	0.375	0.11	1.178	0.262	0.015	0.143
4	0.668	0.500	0.20	1.571	0.350	0.020	0.191
5	1.043	0.625	0.31	1.963	0.437	0.028	0.239
6	1.502	0.750	0.44	2.356	0.526	0.038	0.286
7	2.044	0.875	0.60	2.749	0.612	0.044	0.334
8	2.670	1.000	0.79	3.142	0.700	0.050	0.383
9‡	3.400	1.128	1.00	3.544	0.790	0.056	0.431
10‡	4.303	1.270	1.27	3.990	0.889	0.064	0.487
11‡	5.313	1.410	1.56	4.430	0.987	0.071	0.540

*Bar numbers are based on the number of ⅛ inches in the nominal diameter of the section.
†Bar number 2 in plain rounds only.
‡Bars numbered 9-10-11 correspond to former 1" sq., 1⅛" sq., and 1¼" sq. sizes, and are equivalent to those former standard bar sizes in weights and nominal cross-sectional areas.
°Chord of 12 ½% of Nom. Perimeter.

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J. Harry Moore

J. Harry Moore of the engineering faculty of the University of New Brunswick, Fredericton, N. B., and head of J. Harry Moore and Associates, an engineering firm at Fredericton, died recently in London, England.

After graduating from the University of New Brunswick engineering school in 1933 and having received a MS degree from the Massechusetts Institute of Technology, he spent two years studying structural engineering design at Cambridge University, England.

Before joining the faculty of the University of New Brunswick, he had been with the Dominion Bridge Co., Montreal. He was president of the Association of Professional Engineers of New Brunswick.

Mr. Moore had been an ACI member since 1944.

Bertrand H. Wait

Bertrand H. Wait, president of The Wait Associates, Inc., died recently.

After graduating from Cornell University in 1902, Mr. Wait spent several years on New York Subways; five years with the Board of Water Supply on Catskill Acqueduct work and was division engineer on city tunnels when he resigned in 1913. From 1913-1918, as division engineer of eleven counties in New York State, he was in charge of design and construction of many early concrete roads built by the New York State Highway Dept. From 1918-1929, he was Eastern manager for PCA. Since that time, he had been actively interested in the development and promotion of construction materials principally in connection with concrete and asphalt pavements.

Mr. Wait was a member of ACI, ASTM and ASCE.

Richard S. Yant

Richard S. Yant, president of the Omaha Concrete Products Co., Omaha, Neb., died suddenly at his home recently. He was 38 years old and an ACI member since 1937.

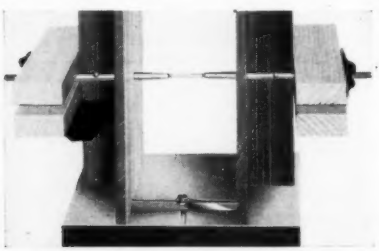
Turner builds Ford plants

Turner Construction Co., New York, and the Long Construction Co., Kansas City, are building a wing section plant for the Ford Motor Co. at Claycomo, Mo., which will have a main one-story building about 1600 x 650 ft and a two-story office section 160 x 350 ft. At peak production 5000 to 6000 will be employed in the plant. Architects of the building are Giffels and Vallet, Inc., Detroit.

At Ypsilanti, Mich., Turner, with the Barton-Malow Co., Detroit, is building a one- and two-story addition totaling 400,000 sq ft to the existing Ford plant there. The addition, estimated to cost $6,000,000, is being designed by Albert Kahn Associated Architects and Engineers, Inc., Detroit.

Warsaw and Wilk elected

David J. Warsaw, president of the Hay-Con Tile Co., Detroit, Mich., has been elected president of the Concrete Products Assn. of Detroit and Benjamin Wilk, Standard Building Products Co., has been elected vice-president.



Williams Clamps showing nail in stud spacing—waler support—and form aligner

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Honor Roll

February 1—August 31, 1951

Newlin D. Morgan, Sr., is still top man on the honor roll with 29 credits. Newlin D. Morgan, Jr., is second with 15½, Eddy Hernandez third with 9½ and J. E. Jellick fourth with 6½. There is still ample time—four more months—to put your name in one of the top spots!

Newlin D. Morgan, Sr. (Ill.)	29
Newlin D. Morgan, Jr. (Mich.)	15½
Eddy Hernandez (La.)	9½
J. E. Jellick (Calif.)	6½
L. G. Farrant (Fla.)	6
H. J. Gilkey (Ia.)	6
Samuel Hobbs (Calif.)	6
Oscar A. Nunez G. (Venezuela)	6
Alexander Klein (Calif.)	4
James A. McCarthy (Ind.)	4
Fernan Rodriguez-Gil (Venezuela)	3½
George D. Youngclaus (Calif.)	3½
Oliver J. Julian (Mass.)	3
Frank Kerekes (Ia.)	3
Alvin C. Loewer, Jr. (Pa.)	3
Jose A. Vila (Cuba)	3
Rafael Ruiz (Guatemala)	2½
Oskar Schreier (N. Y.)	2½
Donald M. Agrimson (S. D.)	2
J. F. Baxter (N. Y.)	2
Robert G. Deitrich (Md.)	2
Frank L. Ehasz (N. Y.)	2
T. C. Kavanagh (Pa.)	2
Thomas J. Reading (S. D.)	2
Chester P. Siess (Ill.)	2
Howard Simpson (Mass.)	2
Harry F. Thomson (Ill.)	2
Ivan M. Viest (Ill.)	2

New Members

41 Individual, 2 Corporation, 11 Junior and 3 Student applications approved for September bring the new membership total to 5370. 17 Applicants from outside the United States indicate increasing world-wide interest in ACI.

Individual

Balcazar Padilla, Luis, Mexico D. F., Mexico (Chief of Strength of Materials Sect., Ministry of Communications and Public Works)

Berson, George L., Raleigh, N. C. (Struct. Engr., L. E. Wooten & Co.)

Bogle, Archibald Gordon, Lower Hutt, N. Z. (Dir., Dominion Physical Lab.)

Bolinger, John C., Jr., Lansford, Pa. (Asst. to Pres., Lehigh Materials Co.)

Courtney, Francis X., Skokie, Ill. (Sales Manager, Material Service Corp.)

Croom, Charles E., Syracuse, N. Y. (Archt. & Prof. of Arch., Syracuse Univ.)

Eubanks, C. E., Knoxville, Tenn. (Chf. Engr., Bureau of Water)

Giordano, Mario, Bronx, N. Y. (Field Office Engr., Corbetta Constr. Co.)

Halsey, Fred H., Texarkana, Ark. (Arch.)

Harris, William G., Pinner, Middlesex, England (Chf. C. E., Admiralty)

Hunt, F. W., Berkhamsted, Hertfordshire, England (Chf. Design Engr., Square Grip Reinforcement Co. (Sunbury) Ltd.)

Karr, Milton, Orange, Texas (Plant Engr., Consolidated Western Steel Corp.)

Keasey, Philip B., Vernonia, Ore. (Highway Engr. Trainee, U. S. Dept. of Commerce, Bureau of Public Roads)

Koo, H. C. Benjamin, New York, N. Y. (Grad. Student, Columbia Univ.)

Lerner, Samuel, Providence, R. I. (Prof. & Cons. Engr.)

Ley, Robert T., Springfield, Mass. (Supervisor, Ley Constr. Co.)

L'Hermite, R., Paris, France, (Dir., Laboratoires du Batiment et des Travaux Publicas)

Majumdar, Manojkumar, Calcutta, India (Asst. C. E., Sir Bruce White, Wolfe Barry & Partners)

Manchester, Otis H., Jr., St. Louis, Mo. (Secy., Treas., Concrete Transport Mixer Co.)

Mason, A. P., Stafford, England (Chf. Engr., British Reinforced Concrete Engrg. Co., Ltd.)

Mazumder, N. K., New Delhi, India (Asst. Exec. Engr., Defense Dept., Govt. of India)

Moser, John, Isle Maligne, P. Q., Canada (Concrete Tech., Aluminum Co. of Canada Ltd.)

Moulton, Wilbur E., Monterey, Calif. (Operates concrete Testing Lab., Normac, Inc.)

Owen, Noel L., Jr., Alhambra, Calif. (Staff Struct. Engr., The Fluor Corp. Ltd.)

Paxton, W. M., Jr., Sheffield, Ala. (Cons. Engr.)

Pratt, Clarence, Lawrence, Mass. (Ashton, Huntress & Pratt)

Randall, Frank A., Jr., Chicago, Ill. (Partner, Cons. Struct. Engr., Frank A. Randall & Sons)

Read, John H., Vancouver, B. C., Canada (Cons. Struct. Engr.)

Ripley, Joseph M., Jacksonville, Fla.

Robertson, LeRoy, Orlando, Fla. (Field Engr., PCA)

Sandoval, Jaime A., Morelia, Mich., Mexico

Sikes, Philip G., West Allis, Wis. (Engr., Wisconsin Electric Power Co.)

Smith, George H., Houston, Texas (Designer, Checker, Job Engr., Giffels & Vallet, Inc.)

Smith, Lomax, Escondido, Calif. (Mgr., Escondido Cement Products Co.)

Tobey, Frank T., Jr., Memphis, Tenn. (Struct. Engr.)

Triche, Roy E., Shreveport, La. (Student)

Visweswaraiya, T. G., Bhadravati, S. India

Whitney, James S., Milwaukee, Wis. (Cons. Engr., Ammann & Whitney)

Wilbur, William E., Los Angeles, Calif. (Chf. Designer, The Fluor Corp. Ltd.)

Williams, William D., Wichita, Kans. (Struct. Design, G. Hartwell)

Corporation

Concrete Assn. of India, Calcutta, India

Meekins, Inc., Hollywood, Fla. (Richard A. Fredericks, Control Engr.)

Junior

Cohen Levy, Salomon, Caracas, Venezuela (Service Engr., Minister of Public Works)

Cusack, Vernon Vere, Cheyenne, Wyo. (Drafting & Design, Wyo. State Highway Dept.)

Dufrechou, Raymond E., New Orleans, La. (C. E., F. P. Dufrechou, Genl. Contractor)

Gillis, Paul, Salisbury, S. Rhodesia, S. Africa (Sr. Struct. Engr., Brian Colquhoun & Partners, Cons. Engrs.)

Huntzinger, Carl J., Douglas, Wyo. (Jr. Engr., Wyo. State Highway Bridge Dept.)

Parker, Martin W., Moline, Ill. (Design and Sales, Austin Crabbs, Inc.)

Patridge, Jesse J., Jr., New Madrid, Mo. (Proj. Engr., Memphis Dist., Corps of Engrs.)

Ponder, Wilburn C., Torrington, Wyo. (Grad. Student)

Schuster, Arthur M., Washington, D. C. (Struct. Design, Ronald S. Senseman, Arch.)

Stray, Frank P., Chicago, Ill. (Student)

Wamsley, Robert M., Birmingham, Ala. (Detailer, Spitler & Armentrout, Inc.)

Continued on p. 20

New Members

Continued from p. 19

Student

DARDEN, PAUL ALBERT, Auburn, Ala. (Ala. Poly. Inst.)
DIAZ DE VILLEGAS, JOSE L., Havana, Cuba (Havana Univ.)
LAMAR, SIMON, Caracas, Venezuela (Central Univ. of Venezuela)

Ready-mixed concrete pricing studied

The Ready-Mixed Concrete Industry Advisory Committee met recently with officials of the Office of Price Stabilization to discuss industry pricing problems. The committee was immediately concerned with the critical problem of increased costs for cement and transportation where it has become necessary to obtain cement from other than normal sources because of serious cement shortage incident to the defense program. To meet the increased demand it has been necessary for concrete suppliers to obtain cement from distant sources at greatly increased transportation costs and in some cases at higher f.o.b. mill prices for the cement.

OPS officials told the committee that an order was being prepared to give relief for increased transportation costs only. The committee recommended that relief be given for both increased cement and transportation costs.

The committee discussed developing a tailored pricing regulation for the industry and considered industry operating and pricing practices in considerable detail. OPS officials advised that a subcommittee will be appointed to work with OPS on this problem.

ACI members on the committee are R. C. Collins, Warner Co., Philadelphia; Herbert G. Jahncke, Jahncke Service, Inc., New Orleans; Walter M. Keeler, The Walt Keeler Co., Inc., Wichita; Frank L. Kelly, Colonial Sand and Stone Co., Inc., New York; William Moore, J. P. O'Connell Co., Boston; Robert F. Porter, Harry T. Campbell Sons' Corp., Baltimore; and John H. Rudolph, Concrete Supply Co., Inc., Evansville.

Collard joins engineering company

Arthur A. Collard, until recently head of the department of civil engineering at Rhode Island State College, Kingston, has joined the engineering staff of Knappen, Tippetts, Abbett Engineering Co., New York, N. Y.

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Rubber tipped vibrator

Rubber tips, standard equipment on Viber electric, pneumatic and gasoline vibrators, have now been added to that company's laboratory model. These tips are said to be less damaging to absorbent form linings and plaster of paris molds and are recommended by the manufacturer for vibrating test cylinders and beams in the field and laboratory.—Viber Co., 726 S. Flower. Burbank, Calif.

Truck mixer manual

Bulletin No. 51-29, Chain Belt Co., describes new features of the company's line of truck-mounted concrete mixers. Recent changes in equipment include a single strand chain drive, improved transmission, hardened drum rollers, newly designed pivot point, water pump relocation and redesigned water seal.—Chain Belt Co., Dept. PR, 1600 Bruce St., Milwaukee, Wis.

Elevating tower

A new descriptive folder, Bulletin No. 734, describes a new lightweight, low-cost material-handling elevating tower. The manufacturer claims for this new tower an unusually low cost of $12.79 per ft, based on 117-ft height.

The tower has 3000-lb capacity, is panel type, made of tubular steel, and is said to be speedily erected in confined spaces. Consisting of only 11 component parts, the prefabricated panels are connected with integral notch-locking mechanism—requiring no bolts or nuts for braces and girts.—American Tubular Elevator Co., Zelineople, Pa.

Light air hammer

A light hand air hammer weighing slightly over two pounds is claimed to be capable of medium-heavy duty work in general shop and maintenance work. At 120 psi the hammer strikes up to 6000 blows per minute at full power or may be metered to any lower speed or striking power by a simple finger trigger action. The 3-point ball bearing chuck is claimed to lock tools in working position eliminating the hazard of tool ejection.

New literature describing, illustrating and pricing this Model G-200 air hammer and the several tool combinations is offered by the manufacturer.—Salsbury Corp., 1161 E. Florence Ave., Los Angeles 1, Calif.

Color matching service

A laboratory service, based on small samples, developing matching color formulas for concrete to meet difficult standards is offered by a manufacturer of integral colors for concrete.—Smith Chemical & Color Co., 55 John St., Brooklyn 1, N. Y.

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- ▶ Positive reinforcement permanently anchored to and combined with structural concrete.
- ▶ Concrete floors and roofs without forms.

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DISCUSSION

Discussion closes January 1, 1952

Sept. Jl. '51

Earthquake Resistant Design Considerations—R. R. Martel

Comprehensive Numerical Method for the Analysis of Earthquake Resistant Structures—Charles S. Whitney, Boyd G. Anderson and Mario G. Salvadori

Multistory Buildings Designed to Resist Earthquakes—John J. Gould

Lightweight Concrete for Lower Construction Costs—J. A. Murlin

Designing for Continuity in Prestressed Concrete Structures—Alfred L. Parme and George H. Paris

Pumice—Lightweight Aggregate—Leslie I. Neher

Hydration Products Formed in Cement Pastes at 25 to 175 C—George L. Kalousek and Milton Adams

Discussion closes February 1, 1952

Oct. Jl. '51

Advances in Precast Floor Systems—F. N. Menefee

Manufacture and Use of Machine-Made Precast Structural Elements—A. G. Streblow

Tilt-up Construction in Western United States—F. Thomas Collins

Diagonal Tension in Reinforced Concrete Beams—Arthur P. Clark

Coral and Salt Water as Concrete Materials—John G. Dempsey

Load Carrying Capacity of Dowels at Transverse Pavement Joints—Henri Marcus

Title No. 48-14

Tilt-Up Construction Costs*

By F. THOMAS COLLINS and EARL M. BENNETSEN†

SYNOPSIS

Cost data for tilt-up construction are broken down into six components—casting platform, forming, pouring and curing panels, erection and construction of columns—and compared with a general over-all cost for reinforced brick masonry construction. As with all cost comparisons necessary allowances must be made for local conditions and prices and for changes in the general economic structure. A short form is presented as an aid in estimating costs.

INTRODUCTION

Tilt-up construction‡ is economical; it saves material and labor. This economy is its greatest asset, and one important reason for the sudden increase in its use. In determining costs, it is important to explore each operation, segregate material from labor, price each fairly, and then sum up the parts. This will permit concentration on the more expensive operations to reduce unit costs to the point where volume construction at low cost is possible.

To compare the cost of precast concrete walls with other types of building walls it is natural to select a masonry wall of equivalent fire rating, and one with which the general contractor is familiar. The cost data for a 6-in. precast concrete wall which follow are believed to be reliable and representative of the Los Angeles, California, area for conditions existing April 24, 1951. For the average job it is approximately 20 cents per sq ft below the cost of an 8-in. brick wall of equivalent fire rating.

COST COMPUTATION

A typical factory building 80 x 100 x 14 ft to the column seat is used as a basis for quantities. It is assumed that 5½-in. concrete walls§ are used and the over-all height from footing seat to parapet is 20 ft 0 in. Wall panels rest on pad type footings with grooves grouted for wall seats. Columns, 20 ft on centers, are formed in place after erection and plumbing of wall panels. The cost of the complete wall has been determined on the gross area of wall surface including the column area. Table 1 gives cost data for the various items of labor and material required and the following sections amplify the table where necessary.

*Presented at the ACI 47th annual convention, San Francisco, Calif., Feb. 21, 1951. Title No. 48-14 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 3, Nov. 1951, *Proceedings* V. 48. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Members American Concrete Institute, Consulting Engineer, San Gabriel, Calif., and Chief Engineer, William J. Moran Co., Alhambra, Calif., respectively.

‡Collins, F. Thomas, "Tilt-up Construction in Western United States," ACI JOURNAL, Oct. 1951, *Proc.* V. 48, p. 133, describes details.

§A nominal 5½-in. wall screeds to an approximate 6-in. thickness.

TABLE 1—COST COMPUTATION

Item	Material		Unit price	Total cost	Cost per sq ft of wall
Preparing casting platform, 400 sq ft	Bond-breaking agent 250 sq ft per gal. @ .75 per gal. Inserts—2 @ 40/400				.003 .002 .005
Fabrication of panels, 400 sq ft	Forms Anchor inserts 1 type A 2 type B 2 type C		 4.60 .35 2.00	 4.60 .70 4.00	.022
	9.30/400 Reinforcing steel 14 ½-in. ϕ x 20 ft x .67 17 ½-in. ϕ x 23 ft x .67 4 ⅝-in. ϕ x 20 ft x 1.04	 188 262 96		9.30	.023
	Total wt, lb 35.50/400	546	.065	35.50	.089
	Concrete 6.5 cu yd per panel		10.00		.163
	Curing compound				.005
					.302
Erection of panels	Motor crane and crew, per hour Rigging and struts		17.00		.019 .020
					.039
Constructing poured columns	Lumber Plywood 1.5 x 26' 2 x 4 x 4' 2 x 10 x 14' 2 x 12 x 14' 2 x 4 x 2'	39 53 47 56 19			
	Total, bfm (With 2/3 salvage) Reinforcing steel 4¾-in. x 15' 100' ¼-in.	214 71 90 17	.105	7.46	
	Weight, lb Concrete, .56 cu yd	107	.065 10.00	6.95 5.60	
	20.01/400 Shee bolts 200 sets @ .20 ea per month 200 studs Crane, rental, 10 hrs		 .046 7.50	20.01 40.00 9.20 75.00	.0503
	Total 124.20/7200			124.20	.017
					.0673

Casting platform

It is usually expedient to use the floor of the building as the casting platform since it is convenient and involves little additional expense. However, a good floor must be placed on well compacted ground to prevent cracking when the crane is lifting panels. Two inserts are placed in the floor for strut supports for each 400 sq ft panel. Immediately preceding construction of

FOR TILT-UP CONSTRUCTION

Labor	Man hours	Hourly rate	Cost	Labor cost per sq ft of wall	Total unit cost, per sq ft of wall
Spraying				.005	
				.005	.010
Layout	8	2.75	22.00		
Forming	24	2.35	56.40		
Setting bolts and inserts	8	2.35	18.80		
Reinforcing steel	8	2.50	20.00		
Rein. steel foreman	8	2.68	21.44		
Material handling	16	1.75	28.00		
Foreman	8	3.25	26.00		
			192.64		
192.64/6 x 400				.080	
Concreting panels					
Screeding	12	1.75	21.00		
Wheeling	12	1.75	21.00		
Shoveling	12	1.75	21.00		
Finishing	32	2.40	77.00		
Final finishing	8	3.60	28.80		
Foreman	8	3.25	26.00		
Total cost for 2400 sq ft			194.80	.081	
				.161	.463
Rigging	2	2.58	5.16		
Carpentry	4	2.35	9.80		
Labor	2	1.75	3.50		
Cement finishing	1	2.35	2.35		
Foreman	1	3.25	3.25		
Labor cost			24.06		
24.06/1200				.020	.059
Forming corners					
Carpentry	64	2.35	150.00		
Forming columns					
Carpentry	112	2.35	263.00		
Stripping columns					
Labor	36	1.75	63.00		
Reinforcing steel					
Iron work	96	2.55	245.00		
Concrete labor	24	1.75	42.00		
Concrete, carpentry	6	2.35	14.10		
Foreman	28	3.25	91.00		
			868.42		
868.42/7200				.120	.187
Total unit cost, solid panel construction, not including profit and overhead					.719

panels bond-breaking agent is sprayed on the floor to prevent bonding of wall panels to floor.

Fabrication of panels

It is assumed that the panels of the typical building have no openings. The effect of openings on cost is discussed later. The construction schedule is assumed at 6 panels per 8-hour day. The actual panel size is approximately

19 x 20 ft; wall area covered including columns is 400 sq ft per panel.

Anchor inserts—Type A inserts provide anchorage for lifting bolts, type B anchorage for strut bolts and type C anchorage for joist anchors, ledger bolts and similar connections.

Reinforcing steel—Reinforcing steel varies with the size and thickness of panel. In the building under consideration the following steel is assumed: ½-in. round 16 in. center-to-center vertical, ½-in. round 14 in. center-to-center horizontal and two ⅝-in. round continuous at top and bottom.

Concreting—It is assumed that part of the concrete must be wheeled and that 6 panels will be concreted in a 6-hour day. A 2500 psi concrete with an admixture is considered for cost purposes. Quantity per panel is (19 x 20 x 5.5)/(12 x 27) = 6.5 cu yd. Immediately after finishing, panels are sprayed with curing compound. This is a minor cost and has been shown to be approximately $.005 per sq ft of wall.

Erection of panels

On a typical building 4 panels can be erected per hour. For this job 3 panels per hour is assumed. Equipment consists of crane, rigging, struts and miscellaneous minor items. Travel time (2 hours in this case) must be added to the lifting time to get the hourly crane cost. To lift 18 panels the total crane time is then 18/3 = 6 + 2 = 8 hours for 7200 sq ft. At a rental of $17.00 per hour the unit cost is 8 × 17/7200 = $.019 per sq ft of wall.

Constructing columns

Two men can form a corner column in an 8-hour day and two intermediate columns in the same time. Two laborers can strip and clean forms on 8 columns in an 8-hour day. Two reinforcing workers can make and place column bars for 18 columns in 6 days. To pour columns using a stinger crane 4 laborers, 1 carpenter, and one foreman will require 6 hours.

It is assumed that the column forms may be used three times. Reinforcing steel shown in Table 1 does not include dowels.

Equipment used in the construction of the columns includes shee bolts used in forming and a stinger crane used in placing concrete. In this typical building, 200 shee bolts are required for 18 columns. Allowing 2 hours travel time each way the total crane time is 10 hours.

OTHER FACTORS AFFECTING COST

Because of variations in buildings there are a number of other factors which must be considered in arriving at the construction cost. Among these are:

a. Building site
b. Size and shape of building
c. Engineering design
   1. Thickness of panels
   2. Concrete design strength
   3. Steel requirements
   4. Size of panels
d. Architectural design
   1. Openings in panels
   2. Texture on panel surface

TABLE 2—ADDED COST OF OPENINGS IN PANELS

Equipment and materials	Unit cost	Total cost	Cost per sq ft of wall	Labor	Man hours	Hourly rate		Cost per sq ft of wall	Total cost per sq ft of wall
Inserts for strong backs		4.00		Layout	1	3.25	3.25		
Forms, bfm	.105	6.30		Carpentry	8	2.35	18.80		
		10.30	.0257				22.05	.0551	.081
				Erection					
Crane .019* × 1.33			.0253	Original crew	2	2.50	24.06		
				Additional men			5.00		
							29.06		
				29.06/(800 × .75) .073 − .059* = .014 = Increased cost of strong backing				.0485	.073

*From Table 1

e. Economic conditions
   1. Wages and material costs
   2. Schedule of construction
f. Over-all efficiency, experience and ingenuity

Openings and strong backing of panels

Panel costs are increased considerably when openings are required. Large openings weaken the panel to the point where strong backing is required. Table 2 shows the increased labor and material costs when a 5 x 10-ft opening is provided in a 20 x 20-ft panel. Erection time of strong-backed panels is increased approximately 1/3 and two additional men must be added to the erection crew.

QUICK ESTIMATE

It is sometimes necessary to make a quick estimate of the cost of a precast tilt-up building. For this purpose the preliminary estimating form in Table 3 is helpful. The unit costs shown are those current in the Los Angeles area in April 1951 and will of course vary with changing local and general economic conditions. Nevertheless the items shown are helpful in avoiding omission of important items of construction cost.

TABLE 3—PRELIMINARY ESTIMATING FORM

Description	Quantity	Unit costs	Material and lump sum	Total
1. General requirements				
Engineering		3½–4%		
General supervision		6–3%		
Insurance, taxes and social security		4%		
2. Job organization				
3. Site preparation				
Remove debris				
Survey				
4. Temporary construction				
Services, temporary		$230–350		
Fence, linear ft		1.00		
Break curb				

Continued on p. 202

TABLE 3 (Cont.)—PRELIMINARY ESTIMATING FORM

Description	Quantity	Unit costs	Material and lump sum	Total
5. Foundation and footings				
Pad footings, sq ft		1.35		
Grade beam, linear ft		2.00		
Fill, sq ft		.20		
Rough grading, sq ft		.03		
Dock high footings, linear ft		2.30		
Ditching, linear ft				
Continuous footing, cu yd				
6. Walls and superstructure				
5-in. walls				
Solid, sq ft		.73–.83		
Windows, sq ft		.80–.89		
Special, sq ft				
6-in. walls				
Solid, sq ft		.78–.88		
Windows, sq ft		.86–.93		
Special, sq ft				
Over 6-in. walls				
7. Roof structure, concrete				
Primary structure				
Truss, sq ft		.30–.40		
Rigid frame, sq ft		.30–.50		
Girder, sq ft		.20–.50		
Beam, sq ft		.20–.50		
Secondary				
Beam, sq ft		.20–.35		
Ribbed slab, sq ft		.85–1.10		
Cladding				
Slab, sq ft		.60–.90		
Standard slab, sq ft		.58–.95		
Special slab, sq ft				
8. Roof structure and ceiling				
Wood sheathing, sq ft		.17–.20		
Wood purlins, sq ft		.16–.18		
Wood trusses, sq ft		.27–.40		
Hung ceiling, sq ft		.33–.38		
Canopy, combustible, sq ft		1.60		
Canopy, incombustible, sq ft		2.25		
9. Concrete floors				
4 in. unreinforced, sq ft		.23		
4 in. reinforced, sq ft		.27		
5 in. unreinforced, sq ft		.30		
6 in. reinforced, sq ft		.35		
10. Openings				
Entrance, each		300–500		
Doors (no hardware), each		35.00		
Windows, sq ft		1.45		
Wood sliding doors, sq ft		.95		
Overhead doors, sq ft		1.85		
Rolling steel doors, sq ft		2.50		
11. Sheet metal work				
Skylights, sq ft		1.55		
Toilet vents, each		25.00		
Flashing, linear ft		1.10		
Gravel stop, linear ft		.45		
Globe vents, in. diam.		1.55		
Globe vent bases, each		25–27		
Special				
12. Roofing				
Composition, 100 sq ft		9–11		
Pitch and gravel, 100 sq ft		12–17		
Sheet metal, sq ft		.25–.33		
Membrane waterproof, sq ft		.35		
Other				
13. Painting				
Interior spray (1 coat), sq ft		.08		
Doors and woodwork, sq ft		.15		
Exterior (2 coats), sq ft		.11–.13		
14. Fire protection				
Sprinklers, sq ft		.30–.32		

Effect of Temperature and Surface Area of the Cement on Air Entrainment*

By E. W. SCRIPTURE, Jr., S. W. BENEDICT and F. J. LITWINOWICZ†

SYNOPSIS

A number of factors influencing air entrainment have been previously reported. Two further factors affecting the amount of air entrained in concrete mixtures, temperature and the surface area of the cement have been investigated. The amount of air entrained decreases with increasing temperature, expressed as its volume at standard temperature and pressure. With increasing surface area of the cement the amount of air entrained by air-entraining agents decreases but there is no significant variation when no air-entraining agent is used.

INTRODUCTION

This laboratory has been studying various factors affecting the amount of air entrained in concrete mixes. Earlier results have been published in three previous papers.[1] Comparatively little work has been done on the effect of the temperature of the mix on the amount of air entrained, or at least there have been very few publications on this subject.[2] As far as the relation between the surface area of the cement and the entrained air are concerned, no published results have come to the attention of the authors. This paper, therefore, reports the results of some experiments undertaken in an attempt to evaluate these factors.

TEST PROGRAM

To study the effect of temperature of the mix on air entrainment, two series of tests were carried out at different times. The first series of tests was made with a nominal 6-sack mix and a nominal 4½-sack mix, at three different temperatures, 40, 70, and 90 F. The second series employed a nominal 5-sack mix at 40, 70, 90 and 120 F. All mixes had a nominal slump of 4½ in. Three admixtures were used in both series, Admixture A—a petroleum hydrocarbon insoluble fraction of a coal-tar hydrocarbon extract of pine wood (neutralized), Admixture B—a triethanolamine salt of a sulfonated hydrocarbon, and Admixture C—a calcium lignosulfonate cement dispersing agent. These represent the air-entraining admixtures most widely used. By weight of the cement, .008 percent of Admixture A, .011 percent of Ad-

*Received by the Institute Mar. 18, 1951. Title No. 48-15 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 3, Nov. 1951, *Proceedings* V. 48. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

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mixture B, and 0.5 percent of Admixture C were used. Sand was a local bank sand, and the coarse aggregate was a crushed limestone, for which the physical constants are given in Table 1. Type I cement was used; a different brand for each series. Surface areas and air entrained in the ASTM mortar test for these cements are given in Table 2 and mix proportions for the three series are given in Table 3.

To investigate the influence of surface area on the amount of air entrained, a single Type I cement clinker was secured and ground, with appropriate additions of gypsum, to different surface areas. Ten different areas were produced, ranging from 1200 to 2600 sq cm per g. The amount of air entrained in the ASTM mortar test was determined for each cement, with no addition and with Admixtures A and C. A single series of concrete mixes was run with no addition and with Admixtures A and C, with a nominal cement factor of 5 sacks per cu yd and a nominal slump of 3½ in. The aggregates used were a local bank sand and crushed limestone coarse aggregate, for which the physical constants are given in Table 1. The admixtures used were .007 percent Admixture A and 0.5 percent Admixture C based on the weight of the cement.

TEST PROCEDURE

The concrete mixes were made in batches of approximately 2 cu ft, mixed 3 minutes in a 2½-cu ft tilting drum concrete mixer. Slumps were measured and 6 x 12-in. cylinders were made, cured, and broken to determine compressive strengths, using appropriate ASTM standard methods. Air contents were calculated gravimetrically and determined immediately after mixing, either with a Klein-Walker air meter or with a Washington air meter. Determinations of entrained air in standard mortar mixes were made in accordance with ASTM

TABLE 1—PHYSICAL CONSTANTS OF AGGREGATES

Sieve Size	Surface area Series	Temperature	
		Series I	Series II
	Fine aggregate*		
4	2	3	3
8	14	13	14
16	31	27	30
30	55	50	51
50	86	88	84
100	99	99	99
F.M.	2.87	2.80	2.81
Sp. Gr.	2.60	2.60	2.60
	Coarse aggregate*		
1 in.	0	48	0
¾ in.	23	84	18
½ in.	79	97	78
⅜ in.	96	98	97
No. 4	99	99	99
	7.18	7.81	7.14
	2.61	2.70	2.62

*Percent retained

TABLE 2—PHYSICAL CONSTANTS OF CEMENT

	Type	Surface area, sq cm per g	Percent air, ASTM 185-47T
Temperature series I	I	1660	6.5
Temperature series II	I	1680	6.7

TABLE 3—MIX PROPORTIONS*

Surface area series		
No addition		1:3.13:3.54
Admixture A		1:2.93:3.54
Admixture C		1:3.00:3.67
Temperature series I		
No addition	Nominal	1:3.63:4.10
Admixtures A & B	4½	1:3.46:4.10
Admixture C	sacks	1:3.48:4.24
No addition	Nominal	1:2.27:3.25
Admixtures A & B	6	1:2.10:3.25
Admixture C	sacks	1:2.16:3.35
Temperature series II		
No addition		1:3.04:3.73
Admixtures A & B		1:2.74:3.72
Admixture C		1:2.90:3.88

*Parts by weight.

TABLE 4—TEMPERATURE AND AIR CONTENT

Temp., °F	Percent air				Water-cement ratio, gal. per sack				Compressive strength, psi											
									1 day				7 days				28 days			
	No add'n	Admixture			No add'n	Admixture			No add'n	Admixture			No add'n	Admixture			No add'n	Admixture		
		A	B	C		A	B	C		A	B	C		A	B	C		A	B	C
Nominal 6 sacks—First series																				
40	2.4	8.8	7.2	5.5	5.75	5.17	5.19	4.86	800*	510*	620*	970*	2210	1540	1750	2650	4280	2700	3430	4790
70	1.9	6.4	5.2	4.0	5.72	5.34	5.24	5.13	880	810	1010	1180	3250	2800	3300	4100	4830	4050	4670	5000
90	1.6	5.1	4.0	3.5	5.94	5.52	5.64	5.17	800	780	830	1270	3150	2900	3320	3820	4580	4480	4580	5280
Nominal 5 sacks—Second series																				
40	1.2	6.4	5.7	4.3	6.88	6.24	6.40	5.94	510	500	670	830	2880	2880	2990	4790	4610	4260	4720	6070
70	1.6	6.0	5.2	4.0	6.96	6.10	6.35	6.09	600	690	830	810	3020	3110	3090	4330	4240	4580	4520	5690
90	1.2	4.7	4.4	3.3	7.32	6.60	6.82	6.24	570	670	760	870	2880	3220	3040	4600	4170	4600	4770	5690
120	1.8	4.5	4.4	4.3	8.24	6.65	7.14	6.78	600	620	740	710	3080	3040	2970	4430	4410	4460	4240	5680
Nominal 4½ sacks—First series																				
40	2.7	11.5	7.8	5.5	8.23	6.95	7.12	7.00	50†	75†	70†	70†	730	760	900	1240	2190	1980	2100	2440
70	1.9	6.8	5.6	4.5	8.26	7.20	7.25	7.06	350	460	530	640	1860	1820	2030	2330	3130	3130	3300	3820
90	2.2	6.4	5.8	4.0	8.22	7.55	7.54	6.86	350	320	370	530	2070	1840	2120	3060	3430	2920	3460	3780

*3 days, zero strength at 1 day.
†2 days, zero strength at 1 day.

Method Designation C 185-47T. Determinations of surface area were made with the Blaine air permeability apparatus and converted by factor to Wagner values.

In the two temperature series the desired temperatures were secured by heating or cooling the water and, if necessary, the aggregates. In the first of these series the compressive strength specimens from the mixes made at 40 F were cured at that temperature. In the second series all specimens were cured at 72 F. In the first temperature series the air contents were determined immediately after mixing and again when the mix had reached room temperature, about 70 F, leaving the mix sealed in the air meter until this temperature was reached. In the second series the air determinations were made immediately after mixing and calculated to room temperature by Charles' law.

TEST RESULTS

Results of the two series of determinations made at different temperatures are given in Table 4. The differences in air content in most cases are small. There does seem to be, however, a general trend toward lower air content, predicated on the volume of air at normal temperature (70 F), with increasing temperature. The behavior in this respect of the mixes with no addition and with the three admixtures is similar but the differences are more evident with the admixtures. Considering the volumes of air as of the temperatures at which the mixes were made, there is an off-setting influence in that the same amount of air as measured under standard temperature and pressure conditions results in a higher volume when the temperature is raised above normal. In general, this is in agreement with the findings of Walker and Bloem.[2]

The water-cement ratios of the mixes obviously increase with increasing temperature, but this is simply the effect of greater evaporation and possibly more rapid hydration of the fine portion of the cement at the higher temperatures. The compressive strength results are entirely normal, and what would be expected from the mix and curing temperature. Where the curing temperatures were normal, as in the second series, the effects of either the higher or the lower temperatures of the mixes as mixed are negligible. Where the curing temperature was the same as that of the mix, as in the case of the two groups of tests at 40 F in the first series, a normal reduction in rate of hardening is found.

TABLE 5—SURFACE AREA OF CEMENT AND AIR CONTENT, ASTM MORTAR TEST

Surface area, sq cm per g		Percent air		
Blaine	Wagner	No addition	Admixture A	Admixture C
2200	1200	7.9	—	—
2590	1430	8.2	17.6	26.7
2770	1510	8.3	17.3	25.2
2930	1600	8.5	16.7	24.6
3110	1700	7.8	16.1	23.4
3310	1810	7.1	15.8	23.2
3710	2030	7.1	15.4	22.2
4020	2200	6.3	14.6	21.9
4360	2380	5.6	12.9	18.7
4770	2610	5.4	11.6	19.1

Results of the determinations of air content by the ASTM mortar test for the cements of different surface areas are shown in Table 5. With no addition the differences in air content are rather small and the trend is not well defined, although there is a general indication that less air is entrained with increasing surface area. With Admixtures A and C this tendency is well defined and there is a substantial range in air contents between the lowest and the highest surface areas.

TABLE 6—SURFACE AREA AND AIR CONTENT, WATER-CEMENT RATIO, AND COMPRESSIVE STRENGTH

Nominal surface area, sq cm per g	Percent air			Water-cement ratio, gal. per sack			Compressive strength, psi								
							1 day			7 days			28 days		
	No add'n	Admixture A	Admixture C	No add'n	Admixture A	Admixture C	No add'n	Admixture A	Admixture C	No add'n	Admixture A	Admixture C	No add'n	Admixture A	Admixture C
1200	1.1	4.8	3.3	7.48	6.66	6.49	690	640	900	2330	2230	3010	3500	3060	4210
1400	0.9	3.8	3.5	7.55	6.69	6.38	810	730	1220	2710	2940	3590	3940	3930	4540
1500	1.3	3.4	4.0	7.50	6.71	6.34	880	900	1400	2900	2900	3410	3840	4230	4760
1600	1.1	2.6	3.2	7.60	6.81	6.16	870	900	1400	2830	2950	3540	4080	4240	4520
1700	1.1	2.9	3.0	7.55	6.56	6.35	1010	1040	1800	3360	3440	4030	4400	4240	5010
1800	1.0	3.5	2.7	7.60	6.71	6.34	990	920	1950	3430	3200	4300	4460	4450	5750
2000	1.0	3.0	2.8	7.58	6.71	6.44	1260	1230	2090	3610	3730	4430	4770	4830	5410
2200	0.9	2.0	2.0	7.66	7.00	6.72	1260	1230	2170	3800	4140	4600	4920	5130	5990
2400	1.0	2.3	2.3	7.60	6.75	6.69	1340	1340	2130	3710	3480	4690	4430	5180	5600
2600	0.9	2.4	2.2	7.63	7.04	6.73	1400	1260	2350	3650	4380	4450	5160	5110	5360

The results of the concrete tests with cements of different surface areas are given in Table 6. Without addition the differences in entrained air with surface area are negligible and within the limits of error of the determinations. With Admixtures A and C there is a definite indication that the amount of air entrained decreases with increasing surface area, although there are a few anomalous results which are probably fortuitous.

There is a general indication that the water-cement ratio for a given consistency increases slightly with increasing surface area but these differences are very small. Compressive strengths at 1-day, as might be expected, increase with increasing surface area. This is still true at 7 and even at 28 days, though less marked at the later age.

CONCLUSIONS

1. The amount of air entrained, expressed as its volume at standard temperature and pressure, decreases with increasing temperature. The differences in the case of mixes with no addition are negligible.

2. With increasing surface area of the cement the amount of air entrained by air-entraining agents decreases. Without air-entraining agents there is no significant variation in air content with surface area.

3. For a given cement clinker the early strengths and rate of subsequent strength gains increase with increasing surface area.

4. The results secured with respect to air entrainment do not differ significantly with the air-entraining agents investigated.

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Title No. 48-16

Foundation for a Large Turbogenerator*

By PAUL ROGERS†

SYNOPSIS

A practical method is presented for the structural design of large turbo-generator supports of reinforced concrete. The loads and forces acting on the support are enumerated, their origin is explained and their values are appraised. Computations for one cross bent are presented in detail and the advantages of embedded steel girders to resist horizontal forces, is emphasized.

The method is applicable to the design of foundations for units of 20,000 KW or larger. The general layout and loading arrangement of all makes are similar, only the values of loading and dimensions differ.

INTRODUCTION

The structural part of power plant design involves very extensive work, both in steel and reinforced concrete. No part of a power station is more important than the support for the turbines and generator. Although good engineering practice always requires that structures be economically designed within the limits of safety, extreme conservatism in the case of turbo-generator supports, or as popularly called "turbine foundations," is entirely justified.

The practical design of the foundation for a 100,000 KW unit, operating at 3600 rpm is described. For brevity, the design is confined to the upper portion of the foundation, *viz.*, above the basement floor level. Below this level the foundation is similar to other heavy foundation work. One of the cross bents is completely analyzed, but no attempt is made to show customary computations for beams and columns.

The use of higher mathematics is avoided, and the problems of vibrations, frequency and resonance are analyzed by empirical methods only. It is undeniable that practical considerations and empirical data often are the basis in engineering design in arriving at the optimum for safe and economical structures within reasonable time allowance for calculations. The author is aware of the numerous American and foreign publications‡ dealing with the question of the natural frequency of vibration of structures; but has found that most of them analyze ideal cases with symmetrical layouts and straight beams and columns which seldom occur in practice. An inspection of Fig. 1,

*Received by the Institute Feb. 8, 1951. Title 48-16 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 3, Nov. 1951, *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1952. Address 18263 W. McNichols Road, Detroit 19, Mich.

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‡*Bibliography on Machine Foundations*, Engineering Societies Library, No. 5. RAUSCH, E., "Zur Berechnung von Turbinenfundamenten," *Beton und Eisen*, 1934. PERRIN, M., *Etude des Vibrations*, Institut Technique du Batiment, Paris, 1946. GOLDBERG, JOHN E., "Natural Period of Vibration of Building Frames," ACI JOURNAL, Sept. 1939, *Proc.* V. 36, p. 81. PARIS, ADRIEN, *Cours de Beton Arme*, Dunod, 1950.

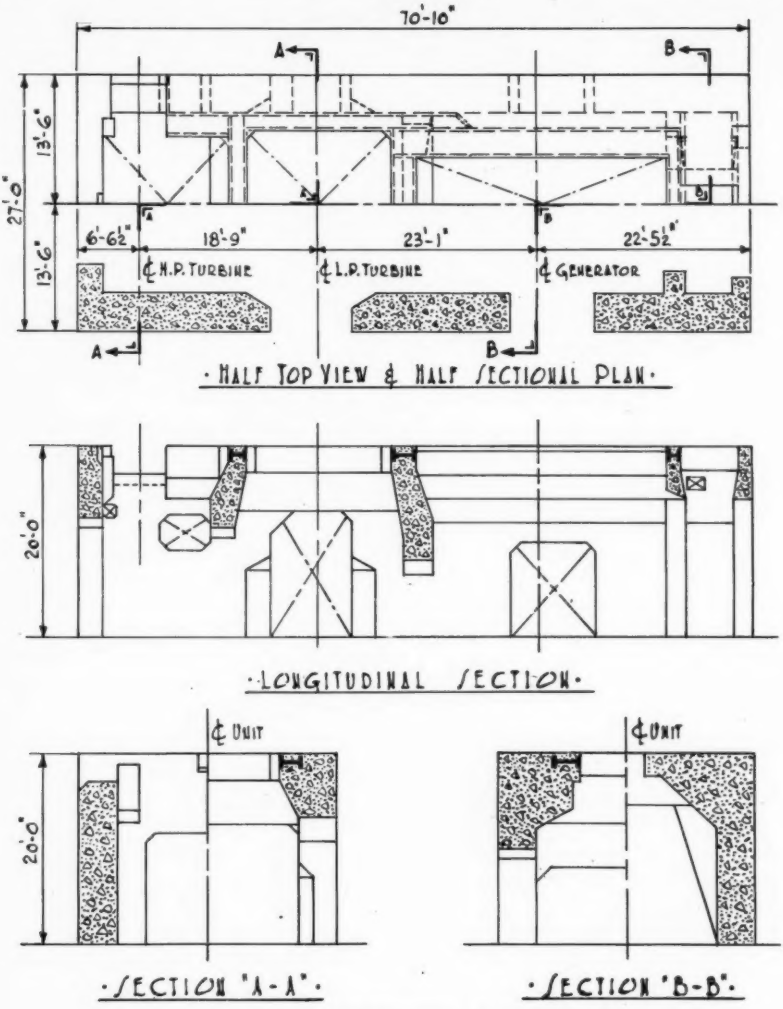

Fig. 1—Details of turbogenerator foundation

which shows the dimensions of an actual turbine foundation, will convince the reader that theoretical resonance calculations for such a support are impracticable, if not impossible. The successful performance of several large turbine supports has proved that excessive vibrations can be eliminated by the simple expediency of maintaining a certain minimum dead load to live load ratio.

Although the design of one particular turbine foundation is described it is applicable to any foundations for units of 20,000 KW or larger. The general layout and loading arrangement of all makes are similar, only the value of loadings and dimensions are different.

The empirical loadings, limits of deflections, impact values, etc., are those recommended by the world's largest turbo-generator manufacturers.

Turbine foundations are subjected to considerable loads and forces. Some of these are the weight of the turbine, generator, condenser, cooling equipment, piping, accessories, etc.; forces due to torque, longitudinal and transverse expansion of the unit, guide bracket reactions, vacuum load, temperature increase, etc.

It has been observed that turbine foundations of sufficient mass are free from observed vibrations. Experience has proved that concrete mass at least equal to two and a half times the weight of all rotating parts, should be provided. Inasmuch as the actual weight of the rotating parts may not be available at the time of the design, it is customary to provide a concrete mass at least equal to one and a half times the total weight of the turbine and the generator. It is important that this ratio shall be maintained for each bent consisting of piers and girders. Furthermore, the concrete mass should be so proportioned that the centers of gravity of the mass and of the unit coincide. If this is impracticable, the resultant eccentricity should be kept at a minimum.

Turbine foundations with very small deflections proved to be very successful. Consequently, it is customary to limit the elastic deformations of the piers and girders to a maximum of 0.020 in. This deformation may be due to flexure or axial load or shear deflection, or the combination of all. This limitation of deflections, however, does not insure against resonance. The relation between mechanical frequency and deflection for an elastically supported concentrated load is expressed by the following equation*:

$$\text{Frequency (Cycles per minute)} = 187.7 / \sqrt{\text{Deflection (in inches)}}$$

According to this equation, the following deflections should be avoided to be secure from resonance under actual loading conditions:

Speed, rpm	Theoretical deflection for resonant vibrations, in.
1200	0.025
1500	0.015
1800	0.011
3000	0.004
3600	0.0028

LOADS AND FORCES

Fig. 2 shows the location of all major loadings and forces.

Load (1)

Weight of the turbine as given on the manufacturers drawings. This is a uniformly distributed load all around the condenser opening. To this an

Turbine-Generator Foundations, GET–1749, General Electric Co.

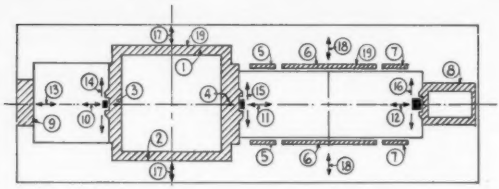

Fig. 2—Loading diagram

impact value is usually added. There is wide variance of opinion among designers as to the impact value to be used; in this practical example 100 percent impact was applied. Thus, (1) is 190 kips plus 100 percent = 380 kips, or $380/4 \times 23 = 4.2$ kips per linear ft.

Load (2)

Vacuum load of the operating condenser. To be considered only if an expansion joint is provided for at the exhaust flange of the turbine. Otherwise, the weight of the condenser is carried partially by the exhaust flanges, and partially by spring supports. In the present case, a vacuum load of 400 kips was computed as a uniformly distributed load, or $400/4 \times 23 = 4.3$ kips per linear ft.

Loads (3) and (4)

Concentrated loads of 100 kips and 19 kips respectively, as given by the manufacturer. Add 100 percent impact.

Loads (5), (6) and (7)

Generator loads of 67, 100 and 67 kips, respectively, as given by the manufacturer. Add 100 percent impact.

Load (8)

Load of the starter, 11 kips in this case. Add 100 percent impact.

Load (9)

High-pressure turbine load of 120 kips. Add 100 percent impact.

Loads (10), (11), (12) and (13)

Horizontal forces of 124, 200, 117 and 60 kips, respectively. These forces are created by the thermal expansion of the unit. It is impracticable to provide absolute frictionless movement of the unit; consequently, horizontal forces, equal to vertical loads times a coefficient of friction, are brought about. There is no accepted opinion about the value of this coefficient of friction, however, and the values recommended by different manufacturers range from 25 to 100 percent. Fifty percent of all equipment load acting upon the cross beams is used in this example.

Loads (14), (15) and (16)

Transverse horizontal forces of 180, 285 and 130 kips, respectively, acting in either direction. The values of these forces usually are given by the generator manufacturer. The origin of these forces requires explanation. During

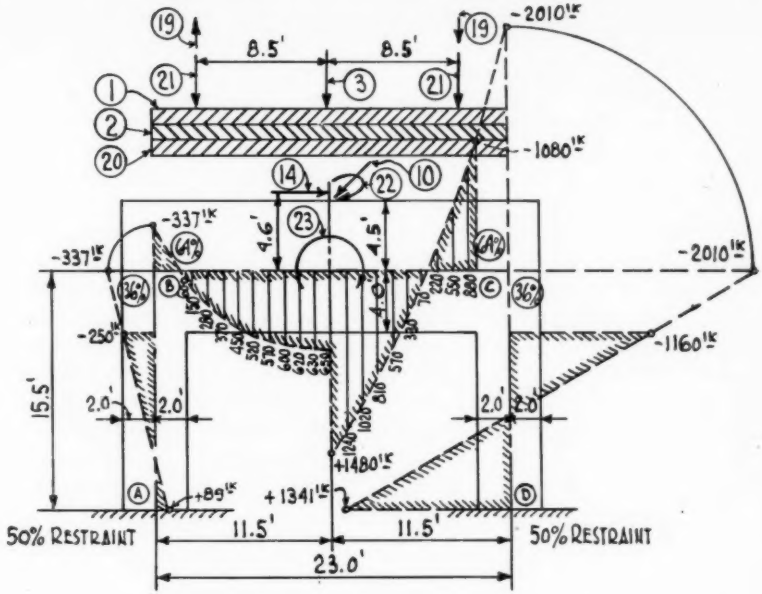

Fig. 3—Dimensions, loadings and moment diagram

the thermal expansion of the unit, the supporting shoes move outward. During a shutdown, the unit cools and the supporting shoes move inward. The friction between the shoes and the plates, however, prevents free movement of the shoes. Movement will thus be retarded and may assume a different amount for each shoe. After several shutdowns, differential movements of the shoes may assume such a magnitude that, if no controls are applied, the unit may be distorted, causing serious misalignment of the shaft. To prevent misalignment, so called guide brackets are firmly attached to the cross girders. These guide brackets maintain the true position of the shaft even under the most adverse conditions.

The forces exerted on the cross beams by these guide brackets are considerable, although their values are sometimes computed with extreme conservatism. In addition, these forces are not applied at the center of gravity of the cross beams, and, in consequence, moments both in the vertical and horizontal planes act upon these beams. To insure perfect alignment, the maximum deformation due to these forces and moments ((22) and (23), Fig. 3) is usually limited to 0.01 in.

To provide adequate anchorage for the guide brackets, it is sometimes required that a thick (2 in. or more) steel plate be embedded in the cross beams, flush with the surface of the concrete. It is possible, and sometimes

economical, to combine this steel plate with an embedded steel girder which is designed to resist any of loads (10), (11), (12) or (13). The narrow space permitted between the high- and low-pressure turbines, or between the low-pressure turbine and the generator, is seldom adequate to develop a reinforced concrete beam within the limitations. It has been suggested by the author* that modern prestressing methods be used for these cross girders. By eccentric prestressing, an elastic camber may be applied to these cross beams horizontally. Under operating conditions, the thermal expansion of the unit would counteract the camber and return the cross beam to a straight position.

Loads (17) and (18)

Transverse horizontal forces of 74 and 117 kips, respectively, acting in either direction. These are also thermal expansion forces, similar to loads (10) to (13). Fifty percent of all equipment load acting on the side girders was used.

Load (19)

Torque load, a twisting force acting on the shaft and produced by the magnetic reaction of the generator which tends to retard rotation. The value of this force may be computed by the following formula:

$$\text{Torque (in ft-kips)} = 7.04 \times (\text{kw}) / \text{Rpm}\dagger$$

In the absence of definite data five times the normal torque is used to take care of possible shock caused by short circuit.‡ This torque is resisted by the side anchorage of the unit, and the resultant couple produces downward force at one side, and upward force at the other side, uniformly distributed along the length of the unit. The computed value of the torque load in the present case is 13 kips (Fig. 3).

Load (20)

Dead load of the girder, 3.5 kips per ft.

Load (21)

Reactions of side beams, 94 kips each.

In addition to these loads and forces, the cross bent shown in Fig. 3 is also subjected to an assumed 50 F temperature rise. The relative stiffness factor of the cross beam is $(2.75 \times 8.5^3) / (12 \times 23) = 6.12$; the stiffness factor (approx.) of the piers is $(10 \times 4^3) / (12 \times 15.5) = 3.42$. The distributing factors are 64 percent for the beam, and 36 percent for the piers.

Maximum reaction, according to loading and final moment diagram is 538 kips. Flexure and axial load per linear ft of pier are $M = 134.1$ ft-kips; $N = 53.8$ kips. Using $f_s = 16{,}000$ psi; $f_c = 975$ psi; $n = 10$; $d = 44$ in.; $d' = 4$ in.; $d'' = 20$-in. and $b = 12$ in., $NE = 224$ and $KF = 312$, compression reinforcing not required. Use minimum $\frac{1}{2}$ percent reinforcing.

DEFLECTION OF CROSS GIRDER

There is no inconsistency between the arbitrary deflection limitation, here

*Rogers, Paul, "Prestressed Foundations," Letter to the Editor, *Engineering News-Record*, Sept. 28, 1949.
†Turbine-Generator Foundations, GET-1749, General Electric Co.
‡Turbine manufacturer's recommendation.

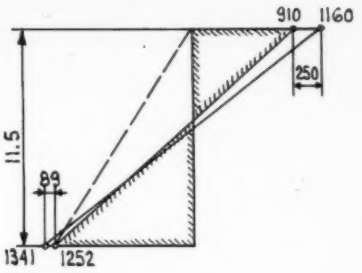

Fig. 4—Moment diagram for horizontal deflection

0.01 in., and resonance deflections. This deflection limitation is insisted upon by turbine manufacturers to eliminate strains and possible misalignment of delicate units. Applying the moment-area method (Fig. 3) only moments on the clear span are used. Positive moments act as loads downward and negative moments as loads acting upward. The left and right reactions of this loading are $R_l = 4940$ and $R_r = 2700$; $x = 9.75$ ft.

M_{max} of moment diagram $= (4940 \times 9.75) - (150 \times 9.25) - (280 \times 8.25) - (370 \times 7.25) - (450 \times 6.25) - (520 \times 5.25) - (570 \times 4.25) - (600 \times 3.25) - (620 \times 2.25) - (630 \times 1.25) - (750 \times .37) = 48{,}200 - 18{,}760 = 29{,}440.$

$I_{approx.} = 2.75 \times 8.5^3 / 12 = 141$ ft^4. $E = 3 \times 10^6$ or 432×10^3 kips per sq ft.

Deflection due to flexure $= \dfrac{29440 \times 12}{432 \times 10^3 \times 141} = 0.0058$ in.

" " " shear $= \dfrac{1.2 \times 476 \times 19 \times 12}{8 \times 23.4 \times 173 \times 10^3} = 0.0040$ in.

Total deflection of cross girder $= 0.0098$ in.

HORIZONTAL DEFLECTION OF CROSS BENT

Horizontal deflection of the cross bent is limited to 0.02 in. The moments acting on the two piers are opposite but not equal. Consequently, the frame will be subjected to sidesway. The resultant moments (clear span only considered) are shown on Fig. 4. By the moment-area method the actual deformation can be computed; $E = 432 \times 10^3$ kips per sq ft and $I_{approx.} = 53$ ft.^{4}

$$\text{Horizontal deflection} = \frac{(1252 \times 5.75 \times 7.67 - 910 \times 5.75 \times 3.82)\ 12}{53 \times 432 \times 10^3} =$$

0.0185 in.

DESIGN OF EMBEDDED STEEL GIRDER

An inspection of the cross beam reveals that the available 21-in. depth is not adequate to resist Load (10) of 124 kips within a deflection limit of .02 in. Consequently, it is advisable to embed a steel girder in the cross beam.

The embedded steel girders and the concrete beams are not designed as composite beams. They serve to take the longitudinal and guide bracket loads only and their use is dictated by necessity and also by the specifications

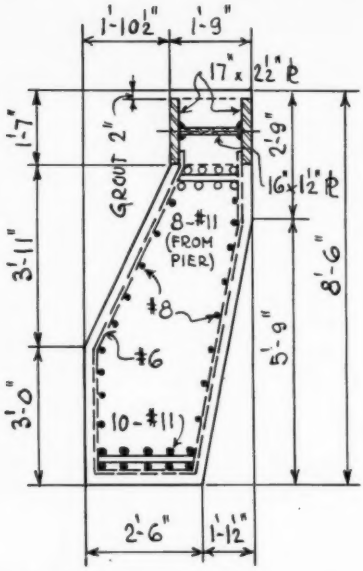

Fig. 5—Cross section of girder between high- and low-pressure turbines

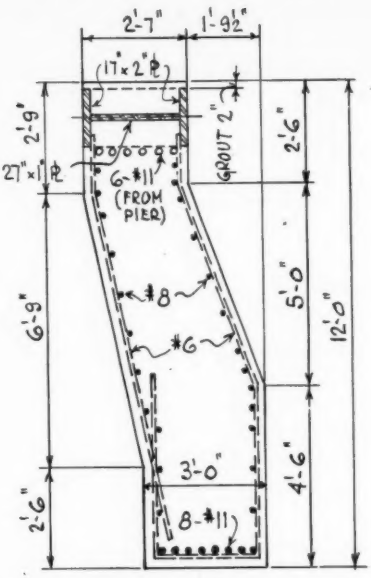

Fig. 6—Cross section of girder between low-pressure turbine and generator

of the manufacturer. Only in the calculations of the deflections of these steel beams were some concrete quantities included because it was felt that the concrete poured around the beams will help resist deformation.

It is assumed that the steel girder is 75 percent fixed, and the maximum allowable deflection is .02 in. (See Fig. 5 and 6).

$$I = \frac{P \times L^3}{144 \times E \times d} = \frac{124 \times 5.83 \times 10^6}{144 \times 30 \times 10^3 \times .02} = 8350 \text{ in.}^4 \text{ (required)}$$

P = 124 kips,
L = 15 ft or 180 in,
L^3 = 5.83×10^6 in.^{3},
E = 30×10^3 kips per sq in.
d (defl.) = .02 in.

Actual moment of inertia furnished:

$$M = \frac{124 \times 15}{7} + 18 = 284 \text{ ft-kips},$$

$$f_s = \frac{284 \times 12 \times 10.5}{9047} = 3800 \text{ psi}$$

I_o of web =	512
I_o of flanges =	45
$Ad^2 = 171 \times 2.5 \times 17 =$	7260
36 in. wide concrete, transformed section =	1230
Total =	9047 in.^{4}

DESIGNING CROSS GIRDER FOR VERTICAL MOMENTS SHEAR AND TORSION (Fig. 7)

M_{max} = 1480 ft-kips (Fig. 3), $d_{reqd} = \sqrt{\frac{1480 \times 12000}{161 \times 24}} = 68$ in.; about 86 in. available.

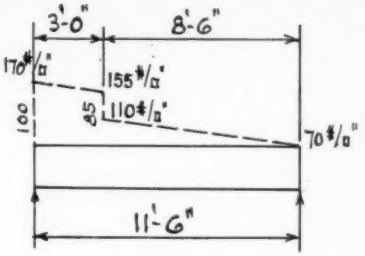

Fig. 7—Shear diagram

$$A_{s\,pos} = \frac{1480}{1.13 \times 86} = 15.2 \text{ sq in. Use 10 No. 11 bars.}$$

$$v_{max} = \frac{538 - 120}{33 \times .87 \times 86} = 170 \text{ psi}$$

Use No. 6 enclosed stirrups 5 in. apart for the first three feet, and 12 in. apart for the remainder. Stirrups to be welded to embedded steel girder. $A_{s\,neg} = 1080 / (1.13 \times 86) = 11.1$, use 8 No. 11 bars, bent in from the piers.

The loads acting on the cross beam do not coincide with the center of gravity of the beam, and thus create torsional stresses. Fig. 8 indicates the loads, torsional moments and stresses. Their values, in this instance, are small, and can be neglected.

The embedded steel girder should have diaphragms between the flanges at close intervals and holes through the web to permit placing of concrete.

GENERAL CONSIDERATIONS

The other parts of the turbine foundation, not included in this paper, should be designed similarly to the bent analyzed herein. Torsional stresses, which were negligibly small in this case, usually are of considerable magnitude in the side girders, and require substantial torsional reinforcement.

All openings should have heavy haunches to minimize stress concentrations. The use of cantilevers should be discouraged. The foundation should be entirely separated from the surrounding building construction to avoid

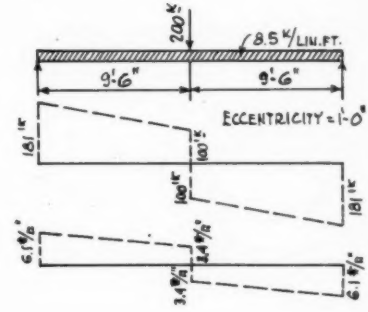

Fig. 8—Torsional loads, moments and stresses

transmitting vibrations. Reasonably low steel and concrete stresses are to be used, and a certain minimum percentage of reinforcing should be maintained.

The supporting sub-foundation should be very substantial, preferably a solid mat several feet thick, and adequate dowels should tie the different parts together. The sketches do not show it, but there is a sub-basement below the so-called foundation, which houses condenser, pumps, steam-pipes, intake-and discharge tunnels, etc. The concrete quantity around and below them is so large that it precludes the influence of vibration between the unit and the subsoil. Experience with large units installed in recent years has led to discontinuing the separation of the turbine and building foundations below the so-called basement level.

To provide true leveling for the unit's bearing plates, a grouting space of 2 to 6 in. is usually provided. Inasmuch as laitance depositing at the surface may be objectionable, it is sometimes specified that the concrete shall be poured higher and chipped down after hardening.

CONCLUSION

The major points in the practical design of a large turbine foundation have been considered, loadings analyzed and one cross bent computed. Due to space limitations many details were omitted, but it is believed that the essential and unusual features in the design of such a foundation have been included.

Discussion of a paper by Paul Rogers:

Foundation for a Large Turbogenerator*

By H. K. FAIRBANKS, ROBERT H. HAWTHORNE and AUTHOR

By H. K. FAIRBANKS†

To cover all the features of design of a reinforced concrete pedestal to support a large turbogenerator is to assume a difficult task. The author has limited his review to the design of one cross bent with comments only on other parts of the structure. The paper gives a good idea of the difficulties encountered when an attempt is made to analyze the stresses and deformations in such a structure. Many attempts have been made by competent designers to evolve a higher mathematical analysis of the stresses in turbogenerator pedestals but the results have not been of significant value. It must be acknowledged that these pedestals are truly indeterminate structures, and while it is possible, as with many such structures, to make arithmetical calculations and obtain answers, the correct proportioning of members must to a large degree be the judgment of the engineer. It is especially true in this matter that the "design" should be placed in charge of only those who have considerable experience to guide them.

The turbogenerator, as a machine, has undergone a great change during the past six to eight years. Within that brief period the size of unit has increased in capacity from 30,000 and 40,000 kw—large units—to a commonplace 100,000 kw, and increasing use of machines of 150,000 kw capability. The physical size and weight has increased in proportion.

Pedestal design has been dramatically changed in the more recent development of the divided condenser, in which half capacity condenser shells are placed alongside and parallel to the unit. This has produced the "low" pedestal which has a deck 16 to 18 ft above the basement floor level, instead of the 30 to 35 ft required for cross condenser settings.

Requirements of the manufacturers for clearances for condenser necks, oil and bleed lines, electrical leads, valves and other items place many difficulties in the way of the designer. We believe it more feasible to lay out the pedestal outline to meet these demands, and then check the members for deflections and shortenings, than to try to establish member sizes by design for most economical section.

*ACI JOURNAL, Nov. 1951, *Proc.* V. 48, p. 213. Disc. 48-16 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

†Supervisory Foundation Engineer, Ebasco Services, Inc., New York, N. Y.

The use of cantilevers, if of substantial section, are not considered by our organization to be objectionable. In fact, in the design of pedestals for outdoor, unhoused, unit installations, side overhangs along the entire length of the pedestal carry the gantry crane used for unit installations and maintenance. In none of the many units of this type installed to date has any trouble developed.

The author is to be commended for his able presentation of a complicated subject.

By ROBERT H. HAWTHORNE*

The author has somewhat over-simplified the problem of design of a turbine foundation in the following remarks contained in his introduction:

"The successful performance of several large turbine supports has proved that excessive vibration can be eliminated by the simple expediency of maintaining a certain minimum dead load to live load ratio."

While it is true that mass properly distributed will in most cases eliminate a great deal of vibratory trouble, the most experienced designers are sometimes chagrined to find similar foundations giving very dissimilar results. This may be due to differences in machine balance, nature of the soil upon which the foundation is mounted, distortion of the structure due to local heating, and resonant characteristics of adjacent structures.

Maintaining a conservative ratio between dead and live loads does not *guarantee* success, but it is a good rule to follow. It is not only desirable to hold vertical and sidewise deflections to small limits, but it should be borne in mind that it is also important that the machine settle uniformly throughout its entire length. The several bearings of a turbine generator must be held to close alignment for most efficient and long-lasting service.

The author has adequately justified the use of structural steel beams to take forces No. 10, 11 and 12 (loading diagram, Fig. 2) where it is impossible to get sufficient horizontal width for a reinforced concrete beam, but there does not appear to be any need for the embedded structural steel members in the longitudinal girders beneath the turbine and generator. Wherever horizontal plates, such as beam webs, are placed directly below heavy load concentrations, there is considerable danger that good bearing contact will not be obtained on the under side of the plate, and a springing condition will result. Deformed reinforcing steel bars can be placed to fully develop the horizontal concrete girders.

If he is proposing to mount a load-bearing surface on a greater depth of grout than 2 in., the shrinkage of even very dry grout would be excessive. Any greater depth than 1½ in. should be filled with steel plates or chocks if it is to hold the loaded surface to true alignment. Grout of greater depths can be used for holding shims and chocks in place.

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AUTHOR'S CLOSURE

Interesting and useful information was conveyed by the discussions of Messrs. Fairbanks and Hawthorne. It is pointed out by Mr. Hawthorne that in some (rather remote) cases the maintenance of proper dead load to live load ratio did not guarantee vibrationless operation. If the machine is not properly balanced, vibration will occur but this can easily be eliminated. Uniform settlement of the foundation is of course mandatory and the science of soil mechanics has reached the stage where office designers may be able to apply it to practical problems.

The embedded steel girders in the side beams were merely provided to obtain proper supports for the cross beams. They have been omitted on subsequent units because by properly anchoring the steel beams to the concrete, the side beams were deemed unnecessary.

The author wholeheartedly agrees with Mr. Fairbanks that only experienced engineers should design such important structures as a turbine foundation. With the shortage of engineers as it is, however, it is not always possible to assign the most experienced men to such problems. It is believed, therefore, that the practical solution of complicated structures ought to be widely publicized, thus enabling the younger engineer to benefit from the experience of others.

The discussions have brought out practical suggestions about grouting, the use of cantilevers, which should be very deep if used at all, and about the use of divided condensers. The inclusion of these discussions have rendered the original paper more complete.

Title No. 48-17

Measurement of the Distribution of Tensile and Bond Stresses Along Reinforcing Bars*

By R. M. MAINS†

SYNOPSIS

Tensile and bond stresses were measured along reinforcing bars by a new technique which does not disturb bond stresses. Curves for representative beam and pull-out specimens show tensile force distribution, bond stress distribution, applied moment versus measured bar tension, comparison of beam and pull-out tensile force distribution, and applied load versus slip of the bar. Building code requirements imply that bond stress in a beam is a direct function of shear, and that longitudinal distribution of bond stress in a pull-out specimen is uniform. These tests show that these are oversimplifications of the problem, which has been understood (but without experimental proof) for some years. Comparison of ordinarily calculated bond with measured local maximum values shows the calculated values to be frequently less than one-half the values measured in these tests. The effect of standard hooks on the behavior of plain and deformed bars is shown for the particular specimens used.

Evidence is presented that cracks in beams decisively affect the magnitude and distribution of tensile and bond stresses—probably one of the more significant results of these tests. Tensile forces in reinforcing bars in beams as ordinarily calculated and as measured in these tests are compared. Calculated values are usually lower than measured values for loads near the ultimate when shear as well as moment acts on the beam.

INTRODUCTION

Previously reported measurements of steel stress in reinforced concrete have been largely of two types: (a) measurements with mechanical gages of rather long gage length which gave average, rather than localized peak stresses, or (b) measurements with short-length electrical gages attached to the exterior of the bar, so that bond was partially destroyed at the very points where strains were measured. The new technique used here overcomes both these disadvantages and furnishes a much more detailed picture of undisturbed steel and bond stress distribution than heretofore.

Previous tests and concepts of bond action deduced therefrom are briefly summarized by T. D. Mylrea.[1] Gilkey, Chamberlin, and Beal[2] give a detailed accounting of their tests as well as a chronological list of references on bond

*Received by the Institute Feb. 19, 1951. Title No. 48-17 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 3, Nov. 1951, *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

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TABLE 1—SUMMARY OF TYPES OF SPECIMENS TESTED

Type No.	Specimen Numbers	Reinforcement	Style of Tests	Hooked or Straight	b in.	d in.	Overall Depth in.	Overall Length in.	Reaction Span in.	Distance Reaction to load in.
1	*P4,5,6,9 P12	1-7/8plain	Pull out	straight	8	9.5	12	21	—	—
	*P1,2,3,7,8 P10,11	1-7/8deformed								
2	*P4H,5H,6H	1-7/8plain	Pull out	hooked	8	9.5	12	21	—	—
	*P1H,2H,3H	1-7/8deformed								
3	*B4,5,6 B10,11,25,26	1-7/8plain	Beam	straight	8	15.5	18	78	72	21
	*B1,2,3,7 B8,9	1-7/8deformed								
4	B16,17,18,27 28	1-7/8plain	Beam	straight	8	10	12.5	78	72	26
	B12,13	1-7/8deformed								26
	B14,15									21
5	B22,23,24,29	1-7/8plain + 4-1/2def.	Beam	straight with bent-up bars	8	10	12.5	78	72	26
	B19,20,21,30	1-7/8def. + 4-1/2def.								
6	*B1H,2H,3H,	1-7/8plain	Beam	hooked	8	15.5	18	78	72	21
	*B4H,5H,6H	1-7/8deformed								
7	X1,2	1-7/8plain	Beam	straight	8	10	12.5	78	72	21
	X3,4	1-7/8deformed								

*Specimens of 1947-48 series - all others 1948-49 series.

action through 1940. In the latest edition of his book, Saliger[3] presents the European point of view on bond action. These various concepts seem to differ but slightly from those discussed by Abrams[4] in 1913, who even then (pp. 192 and 200) recognized the need for measurement of steel stresses over short gage length and at close intervals. The picture of bond action in beams and pull-out specimens which seems to be generally accepted by these investigators is discussed later and those points are indicated where the results of these tests confirm, contradict, or supplement those of previous research.

Determinations of longitudinal distribution of tensile and bond stresses were made on seven different types of beam and pull-out specimens, with approximately an equal number of specimens of each type reinforced with plain and with deformed bars (Table 1). In addition, the necessary control and calibration tests were made for use in interpretation of the test data. Records of crack formation and end slip of bars were made in nearly all cases in which such records were applicable.

SPECIMENS AND APPARATUS

Reinforcement

Main reinforcing bars were all nominal ⅞-in. diameter deformed or plain as noted, bent-up bars ½-in. diameter deformed, and stirrup bars ⅜-in. diameter deformed. All ⅞-in. bars for the pull-out specimens, for beams 1 through 26, and for beams X-1 through X-4 were from the same heat of high-strength steel. Beam No. 30 had a ⅞-in. bar from a different heat of steel intended to be as similar as possible to the previous bars. Beams 27 through 29 had ⅞-in. bars from a stock of AISI-A-3140 steel. The ⅞-in. diameter deformed bars were of the type designated as No. 6 by A. P. Clark[5] while the ½-in. and ⅜-in. bars were type No. 5. Type 6 bars used had an average spacing of deformation

TABLE 2—GENERAL DATA ON PULL-OUT SPECIMENS

Spec. No. #	Type of Bar	f'_c P.S.I.	Avg. Gage Factor $\mu''/\%$	Avg. Steel Area Sq. In.	At 0.001" Slip: Load K.	At 0.001" Slip: Measured Bond Stress P.S.I. Max.	At 0.001" Slip: Calculated Bond Stress P.S.I. Avg.	At Ultimate: Load K.	At Ultimate: Meas. Max. Bond Stress ## P.S.I.	At Ultimate: Calc. Avg. Bond Stress at Failure P.S.I.	Type of Failure
P1	B	3540	59.4	.506	—	—	—	—	—	—	Not carried to failure
P2	B	3540	57.6	.504	—	—	—	—	—	—	" " " "
P3	B	3540	59.7	.503	27.0	—	470	—	—	—	" " " "
P7	B	3600	64.0	.499	not measured		—	45.0	—	780	Bar Fracture
P8	B	3980	64.1	.495	35.5	875	616	50.6	900	875	" "
P10	B	3460	62.7	.491	32.0	1200	555	53.6	1560	930	" "
P11	B	3460	59.0	.522	36.0	—	658	56.3	—	978	" "
P4	A	3490	64.1	.491	9.5	295	165	12.5	460	217	Excessive slip of bar
P5	A	3490	62.8	.500	9.0	—	156	10.4	—	180	" " " "
P6	A	3490	62.8	.500	8.3	—	144	10.2	—	177	" " " "
P9	A	4430	61.8	.510	13.8	375	239	15.4	375	267	" " " "
P12	A	3460	64.0	.492	14.2	—	246	16.9	—	293	" " " "
P13	B	4430	61.6	.487	—	—	—	47.6	—	577	Bar fracture
P14	B	4050	59.2	.491	—	—	—	44.7	1250	541	" "
P15	B	4260	61.2	.493	—	—	—	48.5	—	588	" "
P16	A	4090	62.3	.506	—	—	—	47.9	750	560	Bar fracture
P17	A	3975	62.9	.508	—	—	—	39.1	—	473	" "
P18	A	4380	61.8	.503	—	—	—	46.6	—	565	" "

Notes: Slip measured at free end of bar.

P1, P2, P3 not carried to ultimate.

Slip at free end of bar not measured for hooked bars.

For identification and details of specimens, see Table 1.

Measured maximum bond stress is for the highest load at which readings were taken. This is usually not the ultimate load.

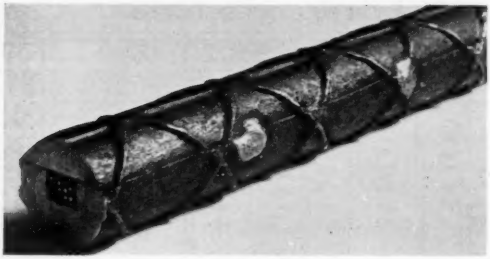
TABLE 3—GENERAL DATA ON BEAM SPECIMENS

Spec. No. #	d in.	Distance Support to Load in.	No. & Type of Bars m	Z Extra 1/4 ϕ in stirrup cage	f'_c P.S.I.	Avg. Gage Factor $\mu''/\%$	Avg. Steel Area Sq. In.	Load in Kips At: 1st Crack	Load in Kips At: 0.001" Slip	Load in Kips At: Ultimate	Bond Stress: Max. Meas. P.S.I.	Bond Stress: Calc. at Failure P.S.I.	Type of Failure
B1	15.5	21.0	1-B	•	3940	64.6	.492	16.0	—				Not carried to ultimate
B2	15.5	21.0	1-B	•	3930	63.1	.492	18.5	—	77.5	1000	388	Bar fracture
B3	15.5	21.0	1-B	•	3980	60.8	.492	17.5	—	71.0		355	" "
B4	15.5	21.0	1-B	•	3980	64.1	.492	19.0	—	74.7		373	" "
B5	15.5	21.0	1-B		3950	63.8	.492	17.0	—	68.5	1000	343	" "
B9	15.5	21.0	1-B		4130	67.9	.492	16.5	—	68.5	1000	375	Compression in concrete-then bar fracture
B6	15.5	21.0	1-A	•	3960	69.5	.492	16.2	28.5	48.5		638	Excessive slip of bar
B7	15.5	21.0	1-A	•	3980	69.5	.492	18.5	29.5	51.8	950	658	" " " "
B8	15.5	21.0	1-A		3980	69.5	.492	16.5	28.5				Not carried to ultimate
B10	15.5	21.0	1-A		4180	69.5	.501	17.0	29.0	[illegible]		415	Excessive slip of bar
B11	15.5	21.0	1-A		4180	64.2	.497	16.5	28.0	31.0		726	" " " "
B12	15.5	21.0	1-A		4180	65.0	.499	17.5	29.0	28.6	1300	700	" " " "
B26	15.5	21.0	1-A		3590	64.8	.500	12.5	25.0	30.6	350	350	" " " "
B13	10.0	26.0	1-B		4000	66.8	.500	7.0	—	34.2	1050	685	Bar yield-then compression in concrete
B21	10.0	26.0	1-B		3760	67.1	.492	6.0	—	34.7	1400	695	" " " " " "
B14	10.0	21.0	1-B		4200	65.1	.503	6.75	—	28.1	1850	787	then bar fracture
B15	10.0	21.0	1-B		4180	65.1	.503	7.0	—	28.1	1850	787	Bar fracture
B16	10.0	26.0	1-A		3930	65.2	.497	5.0	17.5	18.1		361	Bar yield-then compression in concrete
B17	10.0	26.0	1-A		4040	65.2	.497	6.0	18.5	19.7		394	Excessive slip of bar
B18	10.0	26.0	1-A		3980	65.2	.497	6.0	18.5	19.7	400	394	" " " "
B27	10.0	26.0	1-B		4180	64.7	.492	6.0	14.5	16.8	410	336	" " " "
B28	10.0	26.0	1-B		3770	64.6	.506	5.0	16.0	17.1	480	344	" " " "
B19	10.0	26.0	1-#8		4300	67.0	.493	7.0	—	36.0	930	720	Bar yield-then compression in concrete
B20	10.0	26.0	1-#8		3930	67.1	.493	7.0	—	32.0	—	653	" " " " " "
B31	10.0	26.0	1-#8		3840	66.9	.500	7.0	—	35.5	—	724	" " " " " "
B30	10.0	26.0	1-#8		3500	61.6	.505	5.0	—	34.0	870	693	" " " " " "
B29	10.0	26.0	1-#7		4230	63.9	.511	7.0	28.5	38.2	1350	765	Excessive slip of bar-then compression in concrete
B23	10.0	21.0	1-#7		3760	64.4	.503	7.0	29.0	37.8	—	748	Excessive slip of bar-then compression in concrete
B24	10.0	26.0	1-#7		3600	63.8	.501	7.0	32.5	40.5	—	873	Excessive slip of bar-then compression in concrete
B25	10.0	26.0	1-#7		3980	66.0	.510	7.0	27.5	40.7	570	861	Excessive slip of bar-then compression in concrete
B32	15.5	21.0	1-#8##	•	3950	66.4	.492	16.5	—	76.0		370	Bar fracture
B33	15.5	21.0	1-#8##	•	3980	65.6	.493	20.5	—	81.0	1075	394	" "
B34	15.5	21.0	1-#8##	•	3700	66.3	.493	20.5	—	75.0	1000	358	" "
B22	15.5	21.0	1-#8##	•	3330	64.2	.493	16.0	—	66.0	675	893	Bar fracture
B35	15.5	21.0	1-#8##	•	3860	64.7	.493	16.5	—	61.5	—	752	" "
B36	15.5	21.0	1-#8##	•	3890	64.7	.501	16.5	—	59.5	—	729	" "
L-1	10	21	1-#4		4000	—	.498	6.0	14.5	21.0	1000	422	Slip
L-2	10	21	1-#4		3970	—	.498	6.0	14.0	21.0	1000	422	"
L-3	10	21	1-#3		3780	—	.502	4.0	—	14.5	810	435	Yielding then comp.
L-4	10	21	1-#3		3660	—	.500	5.0	—	18.7	860	560	" " " "

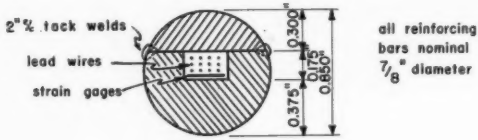
M Stress-strain curves for bar types A - E are shown in Fig. 7.

For identification and details of specimens, see Table 2.

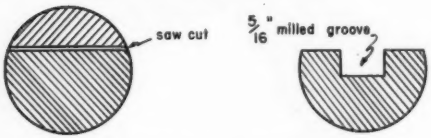
Measured maximum bond stress is for the highest load at which readings were taken. This is usually not the ultimate load.



(a) PHOTOGRAPH OF BAR



(b) DETAILS OF BAR SECTION



(c) STEPS IN FABRICATION

Fig. 1—Method of making test bars

of 0.67 in., an average height of 0.040 in., and a rib width of 0.14 in. Since the bars were rolled prior to the formation of any requirements for the deformations of deformed bars, deformation height was less than the 0.044 in. required by ASTM A305-47T, while other requirements were satisfied. Neither height nor spacing of deformations met requirements of ASTM A305-49. Deformed bars meeting the newer specification would probably show better bond performance than obtained in these tests.

The areas of bars used in the beams are listed in Tables 2 and 3.

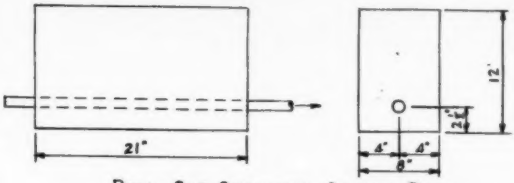
Main reinforcing bars were prepared as shown in Fig. 1 (a) and (b). A longitudinal slice was first cut in a precision band saw (Fig. 1 (c)). The larger slice was then milled to provide a channel 0.312 in. wide by 0.175 in. deep for gages and lead wires. The channel was so cut as to allow the gages to be mounted as near the centroid of the finished bar section as possible, thereby minimizing the effects of local bending of the bar. Paper-backed SR-4 type A-7 gages (¼-in. gage length, 120 ohms) were mounted in the channel with Duco cement. The gages were then baked under an infrared lamp and given a thin coating of cerese wax. The common lead of all gages, of rubber-and-fabric insulated No. 20 solid copper wire, was next soldered in and the joints were insulated.

The remaining lead wires, of No. 22 solid copper magnet wire, were then applied and the joints insulated. Enough cerese wax was finally applied to fill the channel. A resistance between the gage and the bar in excess of 40 megohms was maintained for the whole gage system.

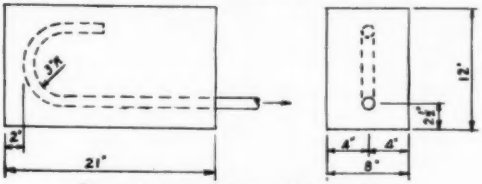
The smaller slice of the bar was next coated with a thin layer of cerese wax along the contact surface and clamped to the larger slice. Tack welds on 2 in. centers held the two parts of the bar together. The welding sequence was arranged to avoid overheating the bar at any time, with cooling periods between passes, so that the gages would not be damaged. For plain bars, tack welds in countersunk holes were later ground flush with the bar surface.

The bars were carefully cleaned to remove wax and grease from the surface before they were cast in concrete.

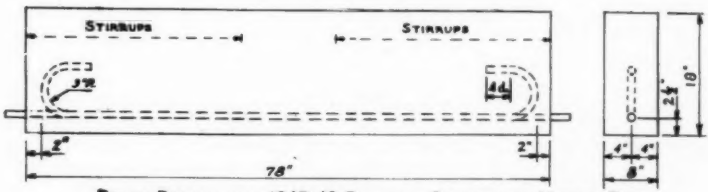
The disposition of the reinforcing bars in the various specimens is shown in Fig. 2. Main bars were positioned by means of holes in the forms through which the bars



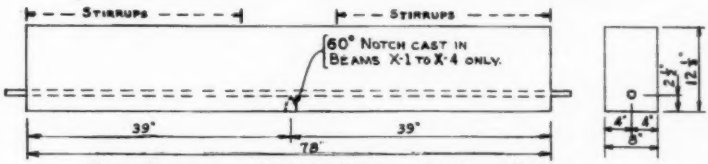
PULL-OUT SPECIMENS, STRAIGHT BARS



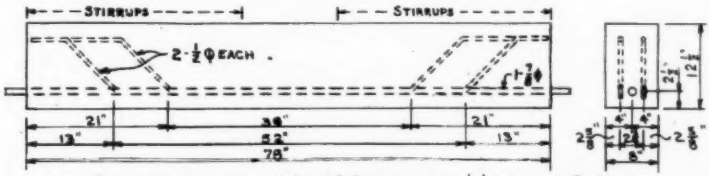
PULL-OUT SPECIMENS, HOOKED BARS



BEAM SPECIMENS - 1947-48 SERIES - STRAIGHT OR HOOKED BARS



BEAM SPECIMENS - 1948-49 SERIES STRAIGHT BARS



BEAM SPECIMENS - 1948-49 SERIES - 4-½φ BENT-UP BARS

Fig. 2—Disposition of bars in specimens

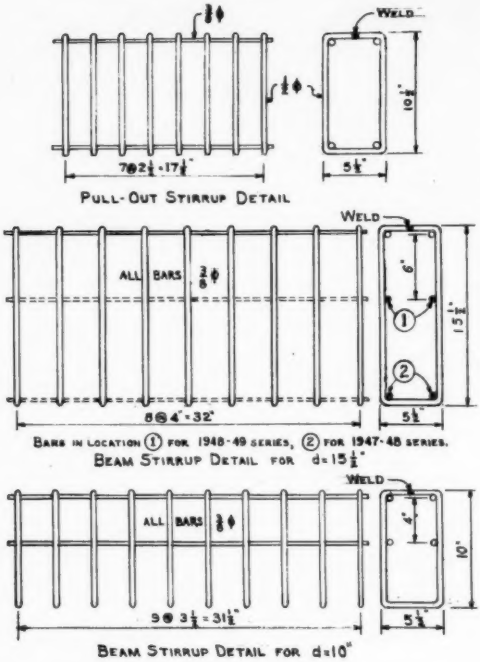


Fig. 3—Stirrup details

protruded. Bent-up and hooked bars were wired firmly to stirrup cages before final assembly of the beam.

Fig. 3 shows stirrup details for the pull-out specimens. These stirrups were needed to prevent longitudinal spliting of the concrete blocks, especially for the deformed bars. Fig. 3 also shows the stirrup details for the two different beam sizes with $d = 15\frac{1}{2}$ in. and with $d = 10$ in. The lower $\frac{1}{4}$-in. bars in the stirrup cages were placed at the bottom corner of the cage in the 1947-48 series, thereby increasing tensile reinforcement over the stirrup length. (For the effect on ultimate load, see Table 3.) Since stirrup cages were fabricated by welding loops to longitudinal bars, these extra tensile bars may have been effective in some unknown way in modifying stress distribution in the main reinforcement. Hence, detailed stress distribution and bond stresses are not given for the 1947-48 series, except for the hooked bar tests.

Control specimens of each type of steel consisted of 24-in. lengths of bar, to provide room for an 8-in. gage length. Some of the $\frac{7}{8}$-in. bars were hollowed out and tested with SR-4 gages as in the regular test specimens.

Concrete

Concrete for the 1947-48 series was mixed in a horizontal drum mixer for five minutes after the water was added. It was placed by shovel from wheelbarrows and rodded in layers. For both pull-out and beam specimens, reinforcing bars were at the bottom of the forms, in the position called "horizontal casting." Control cylinders were taken in pairs for each batch, one from the first wheelbarrow and one from the last (of three), with some additional cylinders taken from an intermediate barrow in several cases.

In the 1948-49 series of tests, the concrete was mixed in a $3\frac{1}{2}$ cu ft tilt-type mixer for 3 minutes after the water was added, and the concrete was transported and placed as before.

In both series of tests, soldered, watertight, sheet-iron lining boxes were used inside wooden forms and were left in place during the entire curing period to inhibit evaporation of water from vertical and bottom surfaces. Exposed top surfaces were covered with a layer of sand, kept wet for three weeks. Cylinders were cured in the same way, in steel forms covered with wet sand, side by side with the corresponding test specimens. A constant temperature room was not available for so many large specimens, but curing was done in a basement room where the temperature variation was usually small.

TABLE 4—SUMMARY OF CALIBRATION OF GAGES

Spec. No.	Type of bar	No. of gages	Gage factor in u''/K			Observed deviation from gage factor $-u''/K$		Percent of deviation observed	
			Max.	Min.	Ave.	Max.	Ave.	Max.	Ave.
*P1	Deformed	9	59.7	59.0	59.4	2.0	0.63	3.4	1.06
P2		10	59.0	53.3	57.6	5.0	1.23	8.7	2.13
P3		8	62.8	57.0	59.7	4.9	1.91	8.2	3.20
P4	Plain	9	65.0	63.2	64.1	4.0	1.00	6.2	1.56
P5		8	63.2	62.2	62.8	1.5	0.62	2.4	0.99
P6		5	63.6	62.0	62.8	1.3	0.54	2.1	0.86
*P1H	Deformed	3	—	—	61.6	8.0	2.54	13.0	4.13
P2H		3	—	—	59.2	5.0	2.55	8.5	4.31
P3H		3	—	—	61.2	7.0	1.64	11.4	2.68
P4H	Plain	3	—	—	62.3	5.0	1.07	8.0	1.72
P5H		2	—	—	62.9	2.0	0.87	3.2	1.39
P6H		3	—	—	61.8	5.0	1.15	8.1	1.87
†B1	Deformed	20	60.3	52.4	56.6	—	—	—	—
B2		20	65.1	53.8	61.6	9.0	1.65	14.6	2.67
B3		20	63.4	55.7	60.8	5.7	1.57	9.4	2.58
B4	Plain	20	64.8	60.8	62.5	4.0	0.95	6.4	1.52
B5		20	64.3	60.9	62.9	7.0	1.16	11.2	1.85
B6		20	64.0	61.0	62.6	3.0	0.86	4.8	1.38

*P—denotes pull-out specimen.
†B—denotes beam specimen, H—denotes hooked bar.

Results of control tests on standard 6 x 12-in. cylinders are listed in Tables 2 and 3. Of the 58 specimens for which control cylinders were tested, the average f_c' was 3790 psi with an average deviation of 280 psi. Eleven specimens had an f_c' under 3500 and 17 specimens had an f_c' over 4000.

Gage arrangements

The distribution of gages along the bars was as shown in Fig. 5–18. Gage lay-out was governed by the maximum number which could be located in the channels. Within this limitation a close gage spacing was maintained in all pull-out specimens and in the portions of the beam-specimens under detailed study, while only a few gages at larger distances were placed in the other parts of the beam specimens.

CALIBRATION OF GAGES

For the 1947-48 tests, most of the straight bars were calibrated in tension with gages in place just before they were cast in concrete, with gage readings taken for various loads. Gage factors in micro-inches strain per unit of total force in the bar were determined, and an average value for each bar was calculated. These values are listed in Table 4, together with the maximum and minimum value for each bar. Deviations of actual individual readings from average gage factor are also shown to give some scale on accuracy.

Hooked bars could not be calibrated in this manner, since hooks had to be formed before gages were applied, which made it impossible to mount the assembled bar directly in the standard grips of a tension testing machine. The general uniformity of the gage calibrations of the straight bars did not seem to warrant a special pulling jig for hooked bars. In hooked pull-out specimens, there were two gages outside and one at the face of the concrete whose readings could not be affected by bond stress. These gages were used as calibration gages for the whole bar, with their average behavior assumed to be representative of the other gages. Table 4 includes deviations and factors observed.

The average behavior of the corresponding three hooked pull-out bars was used to determine the gage factor for the beam bars, with an adjustment for slight variations in area.

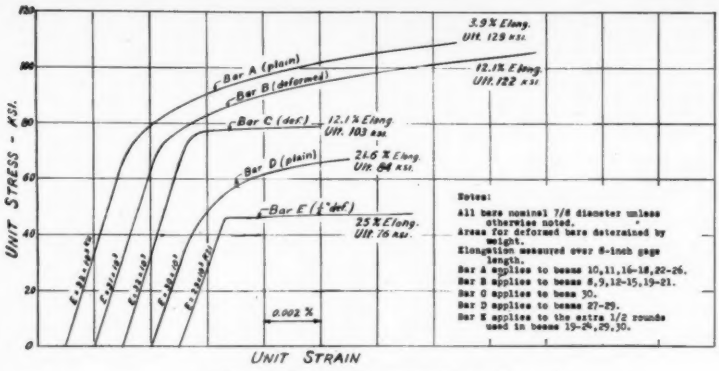

UNIT STRESS - KSI.
UNIT STRAIN
Bar A (plain)
Bar B (deformed)
Bar C (def.)
Bar D (plain)
Bar E (½" def.)
3.9 % Elong. Ult. 129 ksi.
12.1 % Elong. Ult. 122 ksi.
12.1 % Elong. Ult. 103 ksi.
21.6 % Elong. Ult. 84 ksi.
25 % Elong. Ult. 76 ksi.
0.002 %
Notes:
All bars nominal 7/8 diameter unless otherwise noted.
Areas for deformed bars determined by weight.
Elongation measured over 8-inch gage length.
Bar A applies to beams 10,11,16-18,22-26.
Bar B applies to beams 6,9,12-15,19-21.
Bar C applies to beam 30.
Bar D applies to beams 27-29.
Bar E applies to the extra 1/2 rounds used in beams 19-24,29,30.

Fig. 4—Stress-strain curves for steel

In the 1948-49 series, at least two bars of each type were calibrated as before, and the average gage factors were determined for each type. Gage factors for all other bars of the type were determined by taking the gage factors in micro-inches per kip of force-in-bar as inversely proportional to the cross-sectional area.

TEST RESULTS

Control test

Typical stress-strain curves obtained for the reinforcing bars are shown in Fig. 4. Plain bars, A, and deformed bars, B, showed similar behavior, until yielding had progressed to about 0.004 in. per in. strain. For greater strains, the deformed bar showed a much greater ductility, 12.1 against 3.9 percent in 8 in., and a lower ultimate strength, 122 against 129 ksi. Yield strength values at 0.002 in. offset were 86 ksi for the deformed bars and 89 ksi for plain bars.

Deformed bar C was quite different from bar B. Its yield strength was 78 ksi, ultimate strength 103 ksi, and elongation in 8 in. 12.1 percent. In addition, the shape of the stress-strain curve above 65 ksi was radically different.

Bar D, the AISI-A-3140 steel, should have been similar to bar A but was in nearly all respects different. Its yield strength was 62 ksi, ultimate strength 84 ksi, and elongation in 8 in. 21.6 percent.

Bar E represented the ½-in. bent-up bars added to specimens 19–24, 29 and 30.

The above unit stress figures correspond, on the average, to the total force-in-bar values for the different types of bars as shown in Table 5.

Typical stress-strain curves were not much different for 3500-lb and 4600-lb concrete. The 4600-lb curve was somewhat straighter and had a modulus about 10 percent greater. The measured modulus did not equal $1000 f_c'$ nor was it a linear function of f_c'. (The modulus of elasticity was taken as the initial slope of the stress-strain curve.)

Representative specimens

There were more than 270 increments of load (3200 SR-4 gage readings)

TABLE 5—YIELD AND ULTIMATE BAR FORCES FOR DIFFERENT STEELS

Bar type	Size, in.	Total force in bar		Percent elongation in 8 in.
		0.002 yield K	Ultimate K	
A	⅞ plain	44.5	64.5	3.9
B	⅞ def.	43.0	61.0	12.1
C	⅞ def.	39.0	51.5	12.1
D	⅞ plain	31.0	42.0	21.6
E	½ def.	9.0	14.9	26.0

on all pull-out specimens, and more than 720 increments of load (14,000 SR-4 gage readings) on all beam specimens. These readings were reduced to force-in-bar values, most of them were plotted to show distribution of force-in-bar, and over half were translated into bond-stress distribution curves. The results for various types of specimens were studied, particular tests most representative of each type were selected, and these selected results are presented as an abridgement of the entire mass of data. Specimens selected for presentation are those giving as complete a picture as possible in one specimen of what seemed to be the general behavior of the entire group of similar specimens.

A group of force distribution curves and bond-stress distribution curves were prepared for each "representative" specimen. On each force distribution curve, the computed force in the bar is shown as a dotted trapezoidal line for the load increment noted on the line. ("Computed force in the bar" refers to the bar force obtained from the ordinary theory of reinforced concrete behavior. This theory assumes that there is planar distribution of strain across the beam section, that all strains are within the elastic range of both the steel and the concrete, that $n = E_s/1000f_c'$ and that the concrete is incapable of taking any tension in a cracked beam.) In addition for three specimens, force-in-bar versus applied moment curves are shown for each gage in the beams, and force distribution in beams is compared to force distribution in pull-out specimens.

Bar forces were obtained by direct calculation from measured strains in the SR-4 gages, through the use of average gage factors and stress-strain curves from the calibration and control tests. Thus it was possible to determine forces in the bars in both the linear and non-linear range of the stress-strain diagram. Bars for the beams X1 to X4 with controlled crack location, however, were calibrated with special care, and an individual gage factor was determined and used for each gage. This was done so that attention could be concentrated upon interpreting results directly.

Curves of force-in-bar (Fig. 5–18) have been smoothed somewhat when it seemed warranted. When gage spacings for observed values were greater than 3 in., bond-stress values usually were not computed. This was done since a study of specimens X1 to X4 indicated that the principal effects of a crack on bar force could be missed by a gage spacing of 5 in., for example,

TABLE 6—COMPARISON OF REPRESENTATIVE PULL-OUT SPECIMENS

Specimen No.	Type of bar†	At approx. ½ of highest test load*				At highest test load*			
		Load, kips	Meas. max. bond stress, psi	Calculated bond stress, psi	Slip of free end, in.	Load, kips	Meas. max. bond stress, psi	Calculated bond stress, psi	Slip at free end, in.
P4	P-S	6.0	150	76	0.0005	12.5	460	217	0.05++
P8	D-S	24.5	745	310	0.0007	44.5	900+	770	0.0011
P4H	P-H	14.5	275	182	—	29.5	750	370	—
P2H	D-H	17.0	1000	215	—	32.0	1250	405	—

*In this table, "highest test load" refers to the highest load at which complete strain readings were taken, *not* the ultimate load.
†P = plain bar, S = straight, D = deformed bar, H = hooked

and since the location of cracks at the bar surface could not be determined accurately by external observations. The force-in-bar curves may very well not show the true peak of the hump in the force curve unless the crack happened to occur within about ½ in. or less of the gage length. Bond-stress values are, by the same line of reasoning, not as great either in magnitude or variation as they probably would have been had every crack occurred at a gage station.

Method of constructing bond-stress curves

For consistent results standardized technique was used to construct bond-stress curves. The procedure was to:

(a) Plot force-in-bar values to a convenient scale and draw in a reasonably smooth curve between values.

(b) Determine the difference in force-in-bar between two successive gage points A and B. This difference was the total bond force developed in the interval A–B, or

$$\Delta F = F_B - F_A$$

(c) The average unit bond stress in the interval A–B was then

$$u_{ave} = \frac{F_B - F_A}{\pi d_o \times \Delta l}$$

(d) Determine unit bond stress values at points A and B (and intermediate maxima or minima) from the slope of curve (a).

(e) Plot values of u_A, u_B, u_{ave} (and intermediate points when necessary).

(f) Draw in the bond-stress curve so that the particular points, u_A, u_B (and intermediate points) are contained and so that u_{ave} is achieved for the interval.

This procedure gave good qualitative correlation from different persons, and about 5 percent quantitative correlation. Spot checks showed that the area under the bond-stress curve as drawn, as compared with the total difference in measured bar force over the long interval, had an average error of 3.8 percent with a maximum error of 7.4 percent for curves with small bar forces. These errors are considered representative of the bond-stress curves presented.

DISCUSSION OF TEST RESULTS

Pull-out tests

Particular points of interest developed in the pull-out tests are compared

in Table 6 for the various representative specimens. This table, together with Table 2 and Fig. 5-8, shows that:

1. Measured local maximum bond stress regularly exceeds calculated average bond stress by a factor of about 2 or more. This was to be expected, and in itself is not new information. However, the ratio of measured local maximum bond stress to calculated average bond stress is uniformly higher for deformed bars than for plain bars, indicating that deformed bars developed relatively higher bond stresses over shorter lengths of bar.

2. All plain straight-bar specimens failed by slip of the bar at average bond stresses between 180 and 300 psi, while all deformed bar specimens failed by fracture of the bar with no bond failures despite average bond stresses of 800 psi and more. This supports recent changes in bond specifications allowing substantially greater bond stress on deformed than on plain bars.

3. For the plain straight bar, P4, Fig. 5, maximum local bond stress occurred near the unloaded end, and was not reached until maximum total load was applied. Previous investigators concluded that maximum local bond stress would occur near the loaded end of such a bar at relatively low loads, and would move toward the unloaded end as the load was increased, but without any particular change in magnitude of the maximum local bond stress. (See Mylrea[1] p. 524, Gilkey, Chamberlin and Beal[2] p. 84, Saliger[3] p. 83, and Abrams[4] p. 29)

4. For the deformed straight bar, P8, Fig. 6, maximum local bond stress occurred near the loaded end of the bar, while the part of the bar near the unloaded end was never called upon to develop a large bond stress. The deformed bar's behavior was therefore different from the plain bar in this respect. In their Fig. 58, Gilkey, Chamberlin and Beal[2] show a hump of maximum bond stress moving from the loaded end of the bar to the unloaded end as the applied load is increased, but with constant magnitude of maximum bond stress. Fig. 6 shows a similar hump in bond stress for loads 19.5 kips and higher, but the maximum bond stress increases with increasing load.

5. For the plain hooked bars, P4H, Fig. 7, failure changed from slip (as for a straight bar) to fracture of the bar. The hooks provided enough extra anchorage to develop the full tensile strength of the bar.

6. In plain-bar specimens (Fig. 7) the hook was taking roughly ⅔ of the load at fracture, while in deformed bar specimens (Fig. 8) the hook was taking less than ¼ of the load at fracture. Effective bond along the deformed bar was therefore greater than along the plain bar.

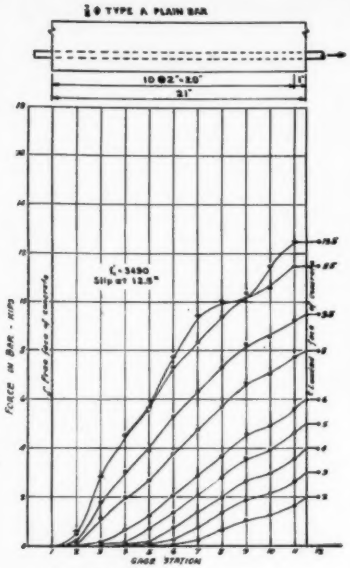
7. Force distribution curves for the deformed hooked bar, P2H, Fig. 8, were somewhat steeper than for the straight deformed bar, P8, Fig. 6. Therefore the hook on the deformed bar increased the bond effectiveness along the part of the bar ahead of the hook. For an applied load of 39.5 kips the hooked bar had 7.5 kips of tension remaining in it 12 in. from the loaded end, or an average bond stress over the interval of 970 psi. The straight bar had 10.1 kips of tension remaining in it at the same distance from the loaded end and the same load, or an average bond stress over the interval of 890 psi. Other intervals and loads show a similar comparison and it is of interest to note that both straight and hooked bars had no tension remaining in them 12 in. from the loaded face for loads in the design range—12 kips or less.

Hooked bars had a 29-in. developed length of embedment as compared to 21 in. for the straight bars. Since this was so, the effectiveness of the hook was not established by these tests *except for the special cases tested.* It seems reasonable to presume that for shorter lengths of embedment the hook would be required to carry more tension.

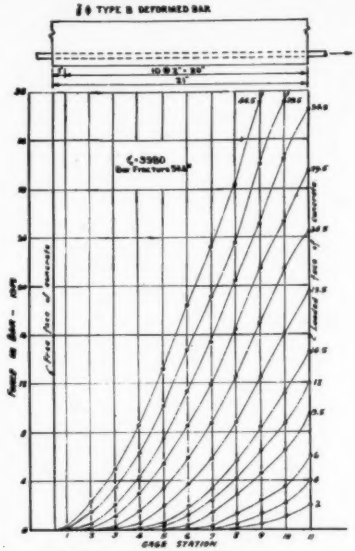
Beam tests

Table 7 gives particular points of interest for the various representative beam specimens. This table, together with Table 3 and Fig. 9–17, shows that:

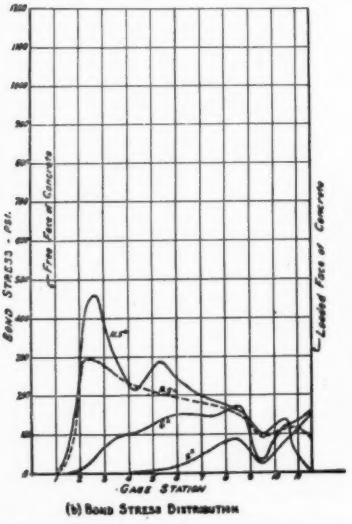
1. For all beam tests, location of cracks governed magnitude and distribution of both tensile and bond stresses. Crack locations were coincident with humps in the



(a) Force in Bar vs. Load

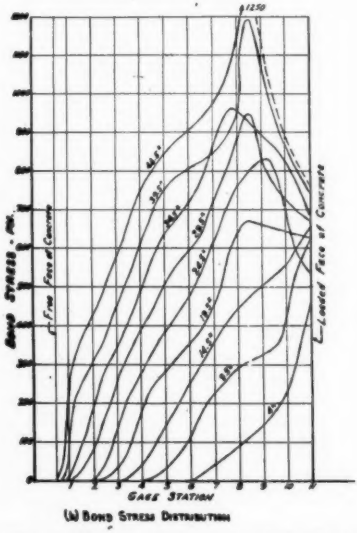


(a) Force in Bar vs. Load



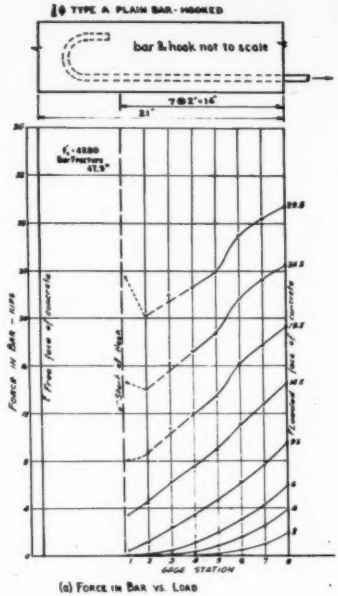
(b) Bond Stress Distribution

Fig. 5—Force in bar and bond distribution Pull-out specimen P4

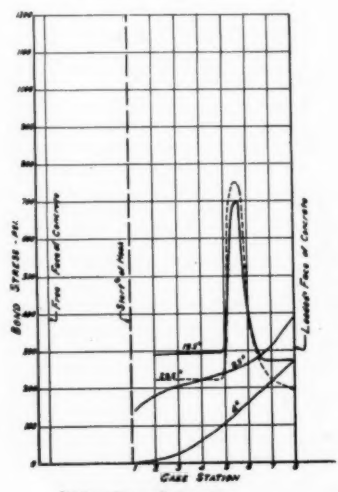


(b) Bond Stress Distribution

—Fig. 6—Force in bar and bond distribution— Pull-out specimen P8

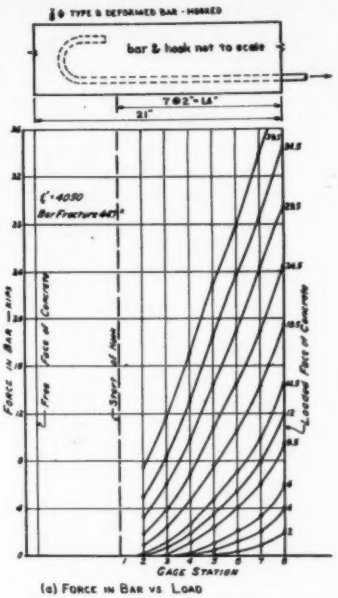


(a) Force in Bar vs. Load

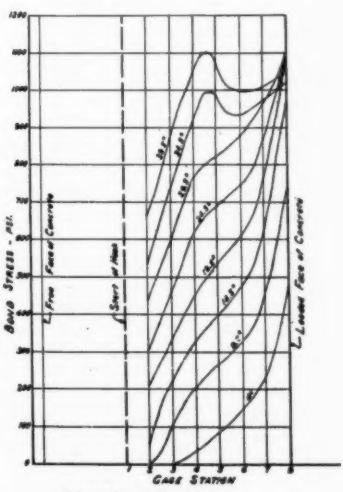


(b) Bond Stress Distribution

Fig. 7—Force in bar and bond distribution—Pull-out specimen P4H



(a) Force in Bar vs. Load



(b) Bond Stress Distribution

Fig. 8—Force in bar and bond distribution—Pull-out specimen P2H

TABLE 7—COMPARISON OF REPRESENTATIVE BEAM SPECIMENS

Specimen No.	Type of bar*	Load, kips	Slip at free end of bar	Bar tension, kips				Bond stress, psi	
				Midway between support and load		Center section			
				Meas.	Calc.	Meas. max.	Calc.	Meas. max.	Calc.
At test load nearest design load for f_s = 20,000 psi									
B25	PS	16.0	0.0018	0.7	5.9	11.2	11.8	220	205
B9	DS	16.0	0.0002	0.7	5.9	8.7	11.8	130	205
B27	PS	6.0	0.0050	0.8	4.2	5.1	8.6	330	120
B13	DS	6.0	0.0002	0.6	4.2	7.1	8.6	680	120
B15	DS	6.0	0.0001	0.5	3.5	5.4	7.0	170	120
B29	PS+	14.0	0.0010	3.9	4.1	7.6	8.2	250	90
B30	DS+	14.0	0.0001	4.6	4.1	8.9	8.2	340	90
B1H	PH	16.0	———	0.6	5.9	12.5	11.8	200	205
B6H	DH	16.0	———	0.4	5.9	2.0	11.8	130	205
At highest test load†									
B25	PS	28.0	0.0076	13.0	10.3	20.8	20.6	1200	355
B9	DS	56.0	0.0032	27.6	20.7	39.8	41.3	1600+	715
B27	PS	14.0	0.0064	12.7	10.3	23.0	20.6	415	275
B13	DS	27.0	0.0020	24.0	19.9	39.1	38.9	3400	540
B15	DS	31.0	0.0008	24.8	18.0	38.0	36.0	970	625
B29	PS+	37.8	0.0497	15.6	11.1	22.5	22.2	570	245
B30	DS+	31.0	0.0005	14.3	9.1	21.5	18.2	600	200
B1H	PH	48.0	———	26.7	17.7	34.4	35.4	560	610
B6H	DH	64.0	———	25.5	23.6	38.6	47.2	560	815

* P = plain bar, S = straight, D = deformed bar, H = hooked, + indicates 4 extra ½ ϕ

†In this table, "highest test load" refers to the highest load at which complete strain readings were taken, *not* the ultimate load.

force-in-bar curves, and also with positive and negative peaks in the bond-stress curves. In beam B-9, Fig. 10, the depression in the force curves between each crack is readily observed, and the tension carried by the concrete is obviously the difference between the bar force at the crack and the lowest bar force between cracks.

2. In Table 7, maximum measured bar tension in the center section of the beam (between loads, where shear was zero) is seen to be close to the value calculated by the ordinary cracked-section theory, especially for the highest load increment. At gages near a crack, measured bar tension sometimes exceeded the calculated tension by 10 percent or more. These results disagree, therefore, with those of McHenry and Walker,[6] who reported test results showing that measured steel stresses in regions of pure bending were always lower than the value obtained by calculation. Saliger[3] consistently shows measured steel stresses as lower than calculated (pp. 152, 166, 168, 365). On the other hand, Fig. 67 in Abrams'[4] bulletin shows measured steel stresses in pure bending in excess of the calculated value.

The curves of force-in-bar versus applied moment in Fig. 11 and 12 for each of the individual gage stations are of interest. Slopes of the curves for the gages in the center section of the beam (where applied moment was constant and applied shear was zero) were quite uniform for any particular beam once cracking was well under way. Since this slope is the reciprocal of the effective *jd*, then the effective *jd* seems to be reasonably uniform for any particular beam in the pure bending section. Force-in-bar versus moment curves, like those of Fig. 11 and 12 were also drawn for all other representative beam specimens. They were found to be similar to those reproduced on these figures and have been omitted from this paper.

3. Also in Table 7 measured bar tension midway between support and load (where shear was high) is seen to exceed consistently the calculated bar tension for the highest load, with the excess varying up to 50 percent or more. This agrees with Abrams'[4] Fig. 67 and 68, but disagrees with McHenry and Walker[6] Fig. 3 and 5, and with Saliger[3] Fig. 337.

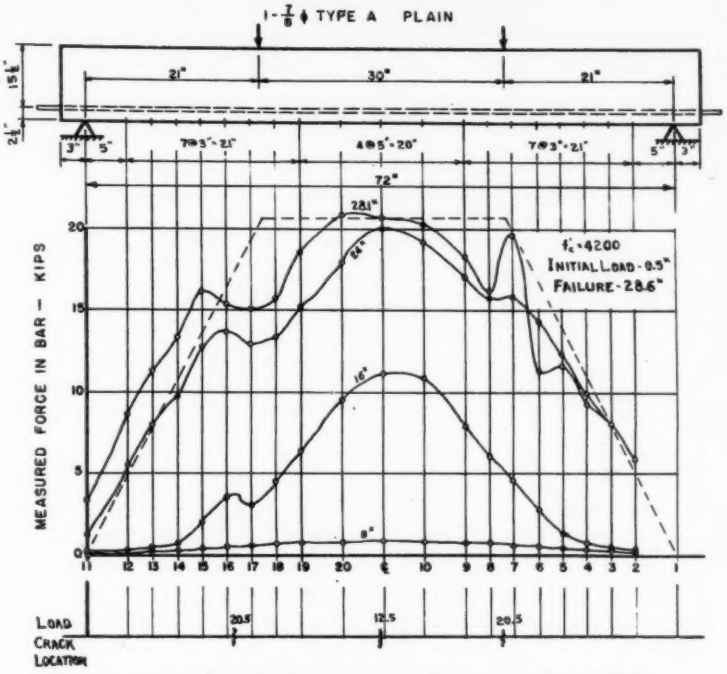


Fig. 9—Force in bar curves and crack locations—Beam specimen B2S

Curves of force-in-bar versus applied moment (Fig. 11 and 12) for gage stations between support and load show a reasonably consistent pattern for each beam. In nearly every case, the slope of the curve becomes steeper for gage stations nearer to the support. This means that the effective *jd* decreases toward the support, and the bar tensions calculated on the basis of constant *jd* are increasingly greater in error with proximity to the support. Saliger[3] shows just such a diminution of effective *jd* in his Fig. 330 for *hooked* bars, and explains this action in terms of arching between the support and the load.

4. Measured local maximum bond stress often exceeds the calculated value by a factor of 2 and more for all loads after cracking was observed (Table 7). This was already stated by Abrams[4] p. 219, and inferred by Mylrea[1] Fig. 6, but on the basis of admittedly inadequate test data (Abrams p. 192 and 200).

5. All plain straight-bar beams failed by excessive bar slip at calculated bond values ranging from 240 to 415 psi. Local maximum bond stresses as high as 1200 psi were measured.

6. All deformed bar beams failed by fracture of the bar (or by bar yielding for the beams with extra bent-up bars), at calculated bond values ranging from 335 to 890 psi. Measured local maximum bond stresses as high as 1800 psi and more were obtained.

7. Plain hooked-bar beams, B1H, Fig. 16, failed by fracture of the bar, with the hook carrying roughly ⅔ of the bar tension at failure.

8. Deformed hooked-bar beams, B6H, Fig. 17, sustained ultimate loads almost identical with the comparable straight-bar beams. The failure was also by fracture of

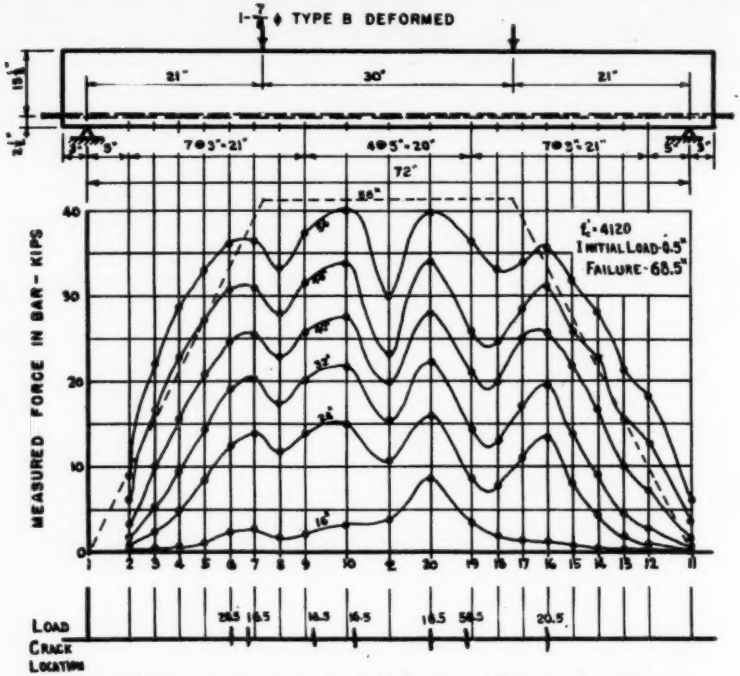


Fig. 10—Force in bar curves and crack locations—Beam specimen B9

the bar, with the hook carrying less than ½ of the bar tension at fracture.

9. For the plain hooked-bar beam, B1H, Fig. 16, calculated force was greatly exceeded by measured force over the part of the bar between the load and the reaction for loads larger than one-half of the ultimate. In fact, the bar force was nearly constant over this portion and only small bond stresses were developed; that is, the bar was "hanging from the hook". On the other hand, the deformed hooked-bar curves (B6H, Fig. 17) were very similar in shape to those for the straight deformed bars. Thus, though the hook enabled the plain-bar beam to carry nearly as much ultimate load as the deformed-bar beam, the manner of carrying that load was entirely different.

10. Two different load locations are reported for otherwise similar deformed straight-bar beams. B15, Fig. 12, had loads applied 21 in. from the supports as for the beams with $d = 15\frac{1}{2}$ in. while B13, Fig. 13, had loads applied 26 in. from the supports as for the beams with $d = 10$ in. Studies were made of the data from these beams to see if the load arrangement difference had any unexpected effect upon either force-in-bar or bond-stress curves, but both beams showed the same behavior in relation to their load patterns. This would probably not have been the case had the difference in load patterns been much greater.

Since the failure of these beams was by tension, the ultimate load for B13 should have been 21/26 as great as for B15 (the ratio of moments in the two beams for the same load). Such was very nearly the case.

Beams with controlled crack location

Force curves for the beams with controlled crack location (X3, Fig. 18)

were particularly satisfactory, since the 36 previous tests had indicated that there would be a local increase in bar force near the crack. It had not been possible to determine previously, because of the gage spacing, just how sharp this increase was and what magnitude of bond stress was associated with it. Since the cracks occurred at the cast notch in the concrete the close spacing of the gages in that region gave the detailed picture needed.

A number of items of interest may be observed from these tests (Fig. 18). First, measured values of force-in-bar do not give a perfectly smooth curve, and the deformed-bar curve was noticeably more smooth than the plain-bar curve. This was the reverse of all previous experience; in other tests, plain bars had always been more uniform in their behavior, and this was considered to be normal. It may have been that, with such close gage spacing, the effect of inherent nonuniformity in concrete could be observed on the plain bars, while the deformed bars helped to mask this effect. Bond stresses deduced from the force curves were in line with previous values for the deformed bar—approximately 800 psi. (The force curves were smoothed before bond stresses were calculated.)

These tests furnish experimental confirmation in detail of the contentions of Abrams, Saliger, Mylrea, and others that very high local bond stresses occur near a crack in a beam. These stresses are independent of shear, but rather depend upon "anti-stretch slip" as Abrams called it, and were obtained in these tests in a region of pure bending. This phenomenon favors closely spaced deformations on a bar to reduce the width of cracks resulting from anti-stretch slip.

Effect of cracks in beams

These tests provide experimental confirmation and numerical data for the effect of cracks upon the force-in-bar and bond-stress distribution in beams.

For a portion of a bar in which the force is reasonably constant (*i.e.*, where there is no shear in the beam) a crack introduces a reasonably symmetrical local increase in bar force. This is caused by the inability of the concrete to carry tension across the crack, so that tension in the concrete is zero at the crack and maximum between cracks. The gradual increase of tension in the concrete with distance from the crack accounts for the decrease of tension in the steel and for the accompanying bond stress between the concrete and the steel. Since this occurs in a region of no shear, it contradicts the common assumption that bond stress is associated exclusively with shear.

For a portion of a bar in which the force is increasing (*i.e.*, where there is shear in the beam) the effect of a crack is again to introduce a local increase, usually unsymmetrical, in the bar force curve. The effect on bond stress is to superimpose positive and negative peaks upon the original constant value.

The measured modifications of force and bond-stress curves by cracks in beams are very similar to those theoretically discussed by Mylrea. Given the load and crack pattern for a beam, it should be possible to sketch the qualitative force and bond-stress curves. However, there is not yet sufficient

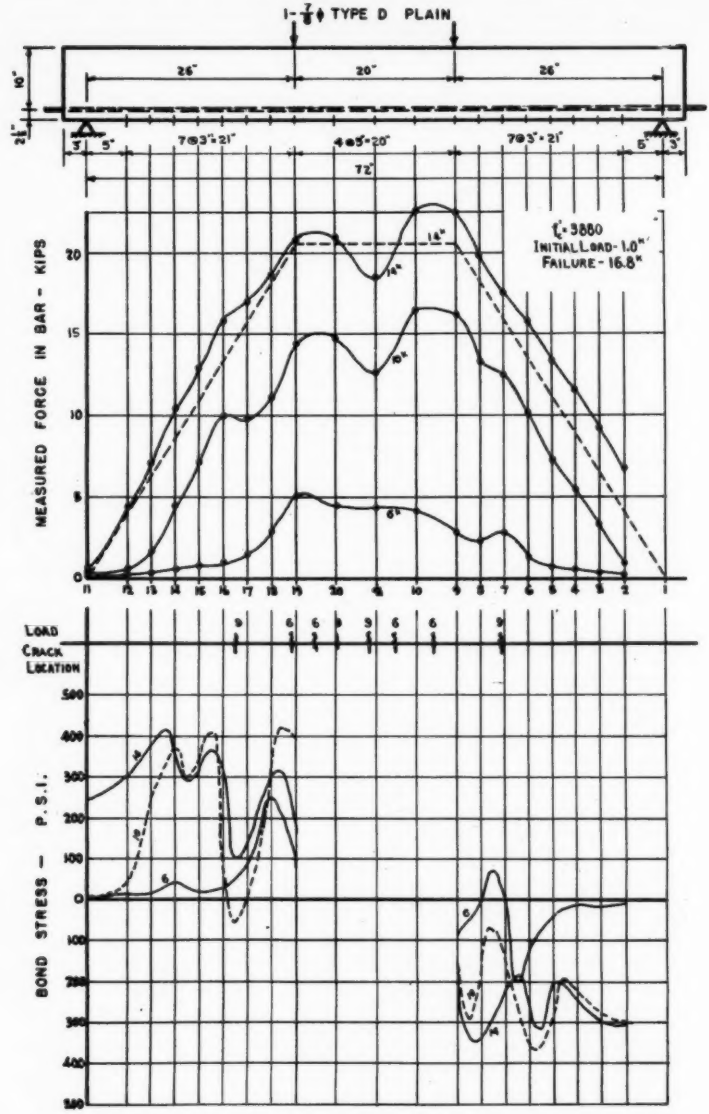


Fig. 11a—Force in bar, bond stress and crack locations—Beam specimen B27

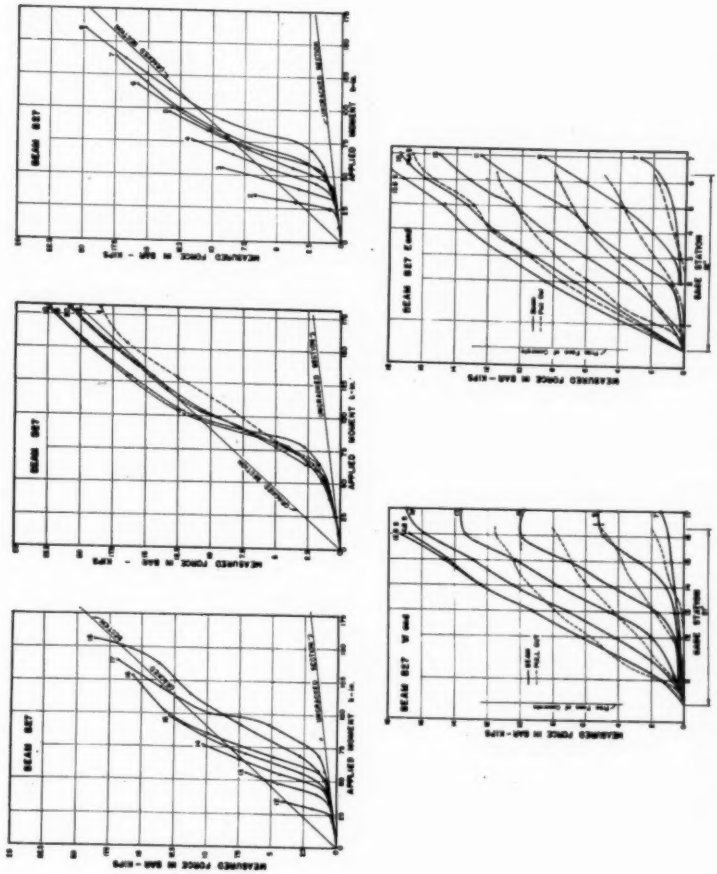


Fig. 11b—Applied moment and gage locations vs force in bar—Beam specimen B27

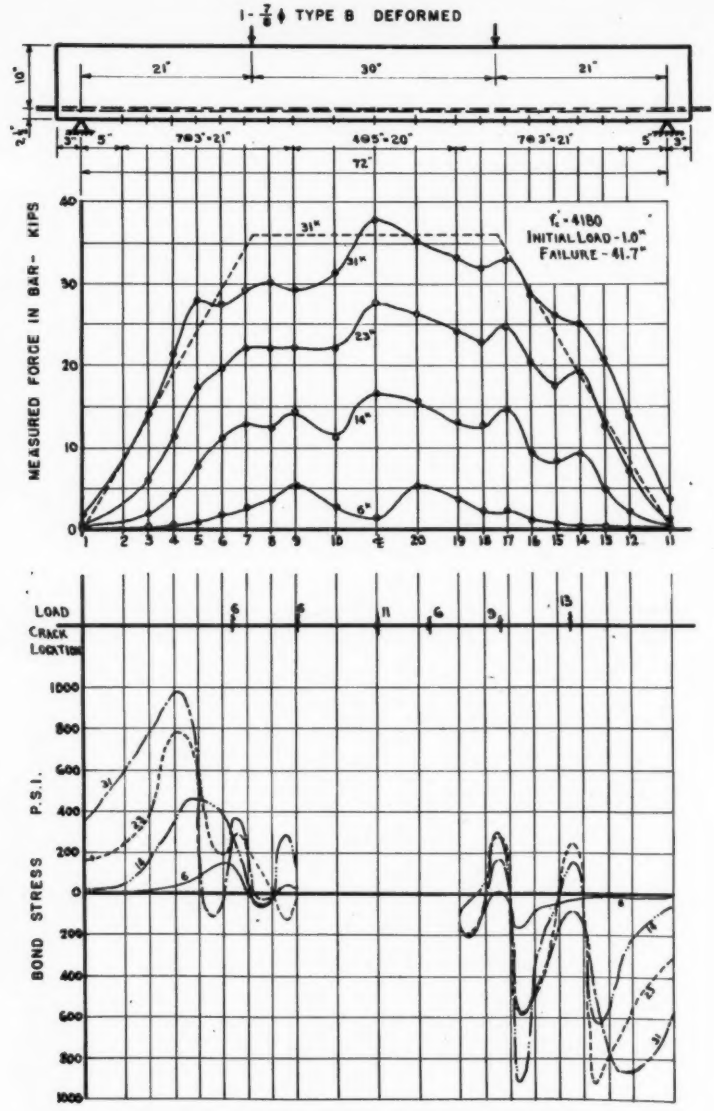


Fig. 12a—Force in bar, bond stress and crack locations—Beam specimen B15

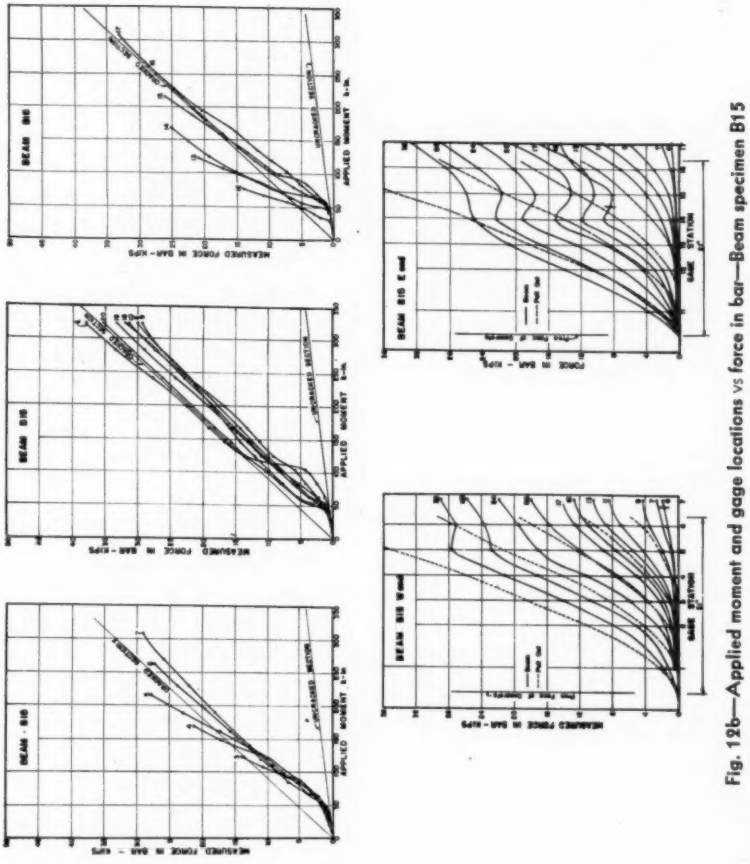


Fig. 12b—Applied moment and gage locations vs force in bar—Beam specimen B15

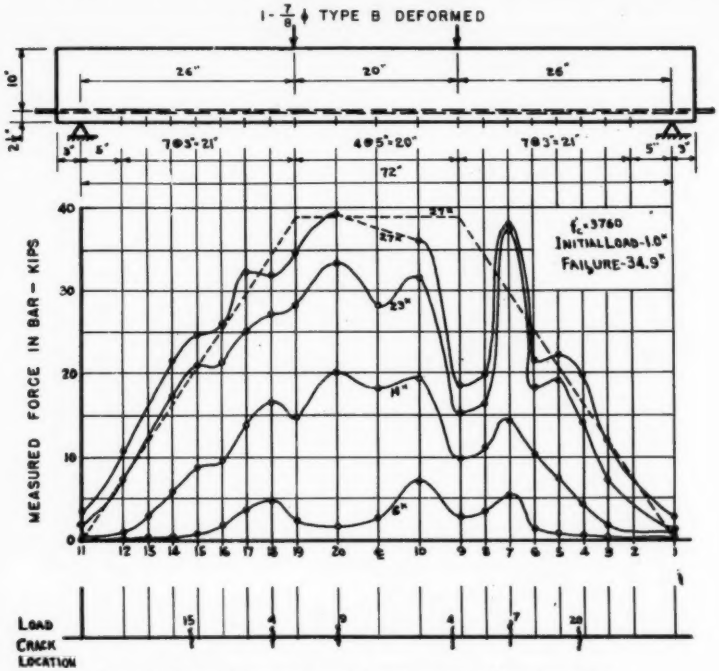


Fig. 13—Force in bar and crack locations—Beam specimen B13

information to allow a quantitative application of these principles, any more than there is yet a rational means of predetermining the crack pattern.

Correlation of beam and pull-out specimens

In each of Fig. 11 and 12 is a set of curves in which the force-in-bar for the beam is superimposed on the corresponding pull-out curves. The shapes of the two curves are markedly similar when thus plotted to the same scale, with the free end of the beam bar coincident with the free end of the pull-out bar. This similarity is limited, however, to the portion of the beam between the support and the nearest crack. For example, because cracks frequently appeared nearer the free end of the beam than the 21-in. length of pull-out specimen, many of the beam curves are higher and steeper than the pull-out curves, but of a shape which seems to be a reasonable development of them. The implication is that there is close correlation between the behavior of the portion of a beam bar between the free end and the nearest crack, and the portion of a pull-out bar between the free end and a point on the bar the same distance from the free end as the crack in the beam. These data seem reasonable evidence of good correlation between the two types of test *with the restrictions above noted.*

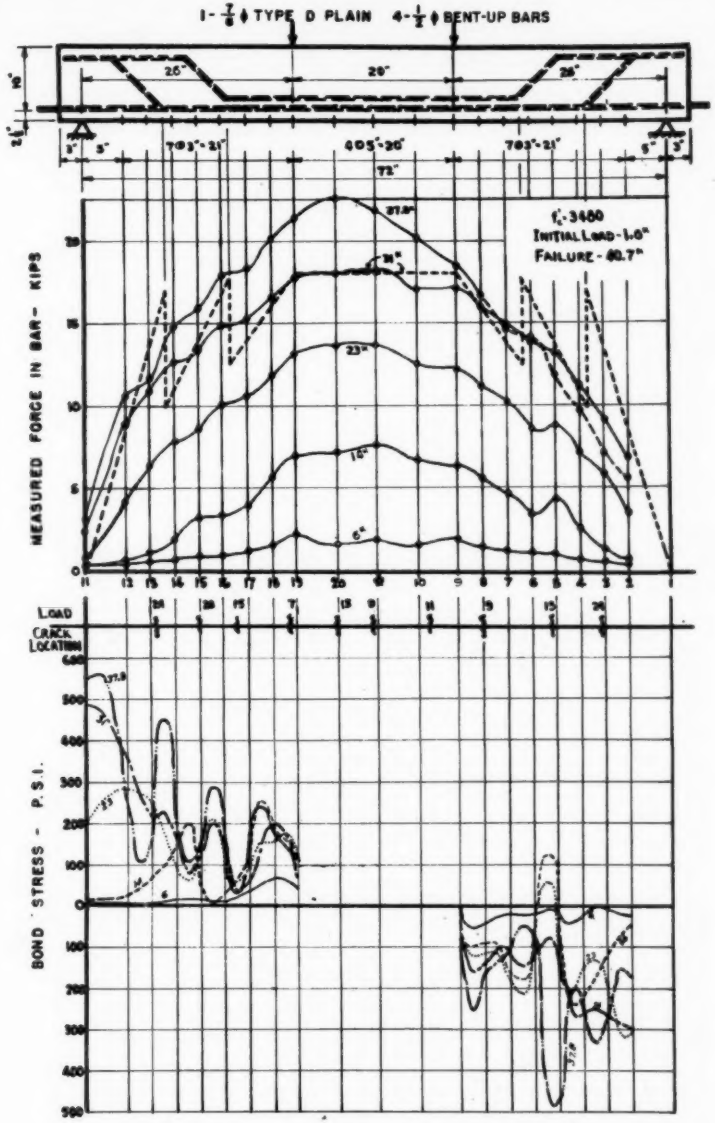


Fig. 14—Force in bar, bond stress and crack locations—Beam specimen B29

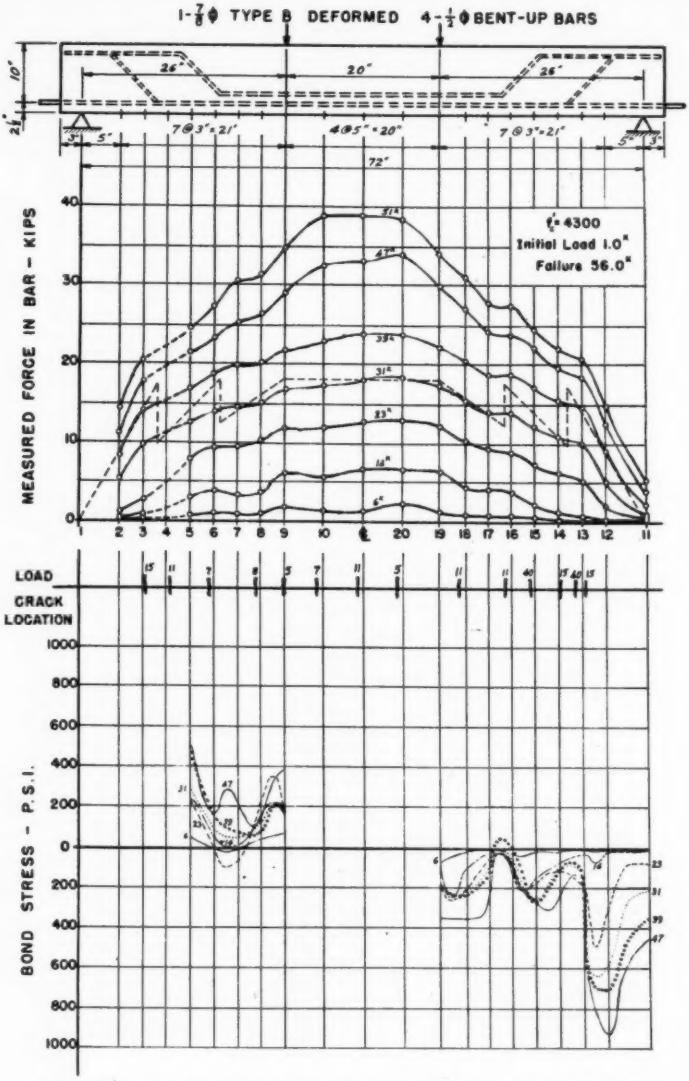


Fig. 15—Force in bar, bond stress and crack locations—Beam specimen B19

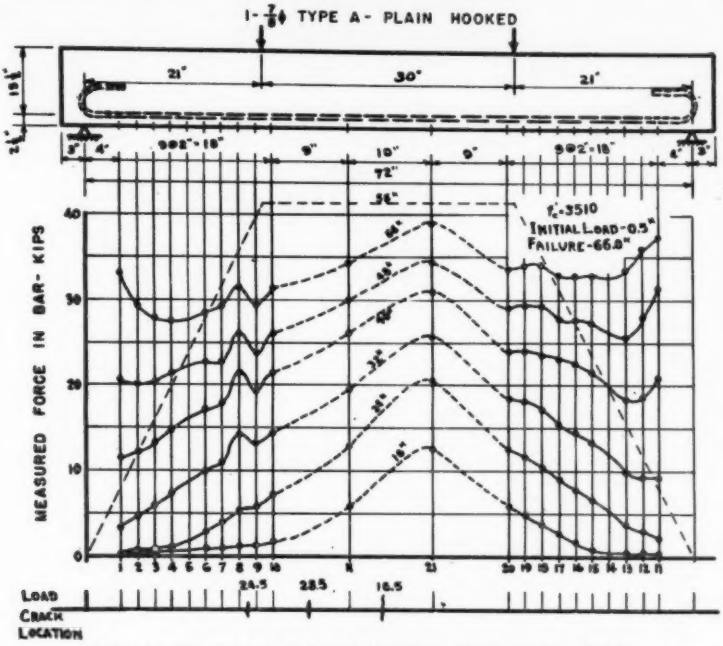


Fig. 16—Force in bar and crack locations—Beam specimen B1H

CONCLUSIONS

1. These tests show that large local bond stresses occur in regions of zero shear, and that total shear and local bond stress are not directly proportional.

2. Usual anchorage requirements treat bond stress as constant in value along a bar under tension such as in a pull-out specimen. These tests show a fairly uniform bond stress only for plain, hooked, pull-out bars in the yield range of the bar.

3. Local maximum bond stresses frequently exceed those calculated by building-code procedures by a factor of 2 or more.

4. Plain bars without hooks, both in pull-out and in beam specimens, failed in bond by excessive slip of the bars at loads between one-third and two-thirds of the yield strength of the bars. The forces in the bar causing excessive slip in pull-out specimens were somewhat lower than in beam specimens.

5. Deformed bars without hooks of the type and of the embedment used in this series developed bond strength high enough to produce fracture of the bars in all specimens, rather than bond failure.

6. The use of hooks on plain bars seemed to make them almost as effective in ultimate capacity as deformed bars, but the plain bars "hang from the

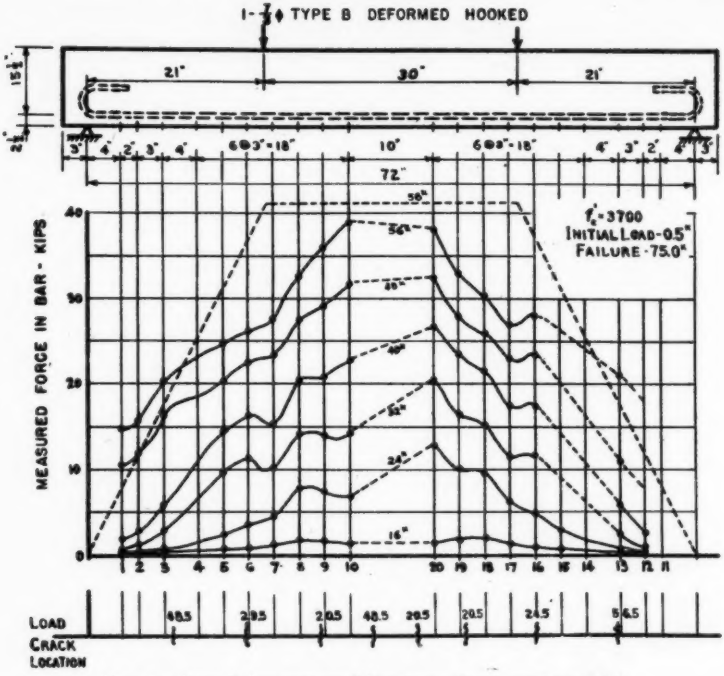


Fig. 17—Force in bar and crack locations—Beam specimen B6H

hook" while the deformed hooked bars developed considerable bond along their straight length.

7. In the part of the beam in pure bending, measured tensile forces agreed reasonably well with the ordinary theory, with measured force sometimes exceeding the calculated value by 10 percent and more. In the parts of the beam subjected to shear as well as moment, measured tensile forces in bars generally exceeded calculated values. Midway between support and load this excess was frequently up to 30 percent and more.

ACKNOWLEDGMENTS

The larger part of this investigation was sponsored at Cornell University by the Committee on Reinforced Concrete Research of American Iron and Steel Institute, represented first by R. R. Zipprodt and later by C. A. Willson. The initial proposal for the study was made by Dean S. C. Hollister.

The tests were performed as part of the work of the Department of Structures, School of Civil Engineering, Cornell University. Dr. George Winter was in administrative charge and contributed helpful consultation and suggestions on technical matters. Milton R. Martinez[7], F. J. Malkmus, and Chwin-Jen Peter Sih assisted in making the tests and preparing the data.

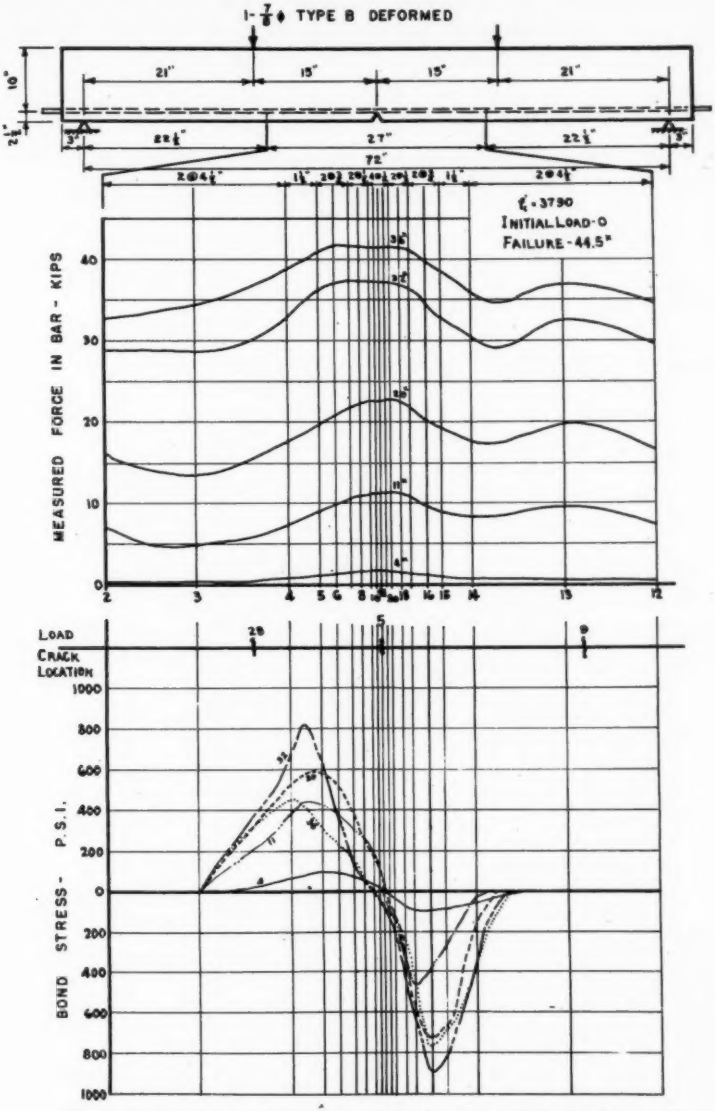


Fig. 18a—Force in bar, bond stress and crack locations—Beam specimen X3

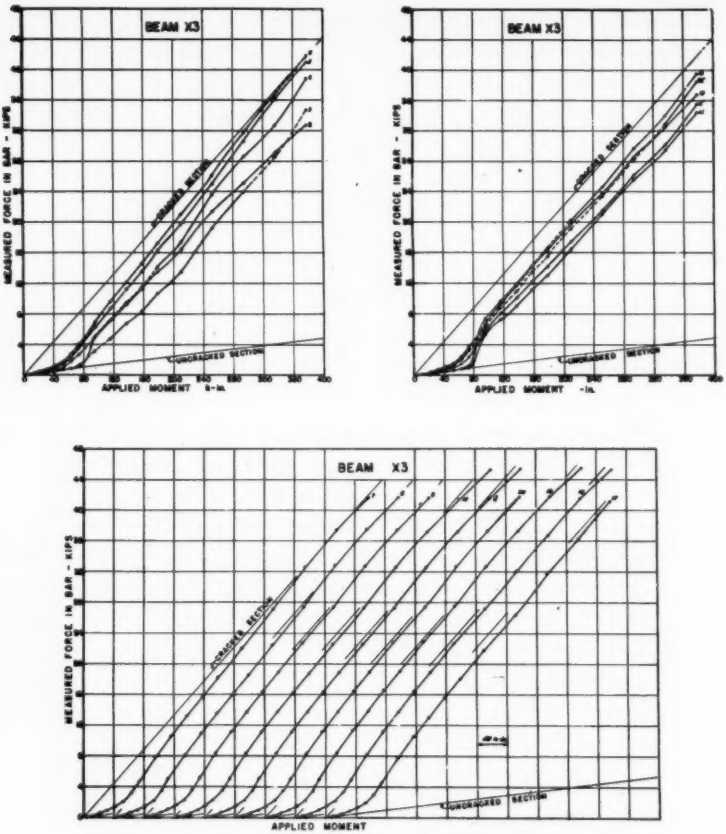


Fig. 18b—Applied moment vs force in bar—Beam specimen X3

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Discussion of a paper by R. M. Mains:

Measurement of the Distribution of Tensile and Bond Stresses along Reinforcing Bars*

By A. D. HOGG and L. SCHENKER, ARON L. MIRSKY and LYLE K. MOULTON, RAYMOND C. REESE and AUTHOR

By A. D. HOGG and L. SCHENKER†

A rational comparison of the performance in bond of plain and deformed bars requires a thorough understanding of bond stress distribution along reinforcing bars. Although much valuable work has been done of late in comparing bond strength of plain and various types of deformed reinforcing bars, this has resulted chiefly in the comparative rating of the bars according to their average bond strength. Professor Mains' report does much toward filling a gap in our knowledge on the nature of bond.

The writers feel that rather than contradicting the "common assumption that bond stress is associated exclusively with shear," the test results emphasize a recognized limitation of the simple reinforced concrete theory, and provide interesting quantitative data on the extent of the departure from the simple theory. It is still true that bond and shear are related, though in the vicinity of a crack they are in nature, but not in magnitude, of a secondary order. Incidentally, another example of a case where bond stress may exist without external shear was mentioned by W. H. Glanville in 1930‡: in an unloaded reinforced concrete beam, shrinkage and temperature changes produced relative strain between steel and concrete, resulting in bond stress.

The writers were particularly interested in the tests with hooked bars, presenting in a convincing manner experimental proof of the long-held contention that plain bars "hang from the hook;" not so with the deformed bars. Professor Mains has shown that in a pull-out test with a hooked deformed bar of typical dimensions, the hook carries less than ¼ of the load at failure, whereas the corresponding value for a plain hook is ⅔ of the load. This confirms the superior bond performance of deformed bars and that hooks are not needed with deformed bars except, perhaps, in some special cases.

Attention should be drawn to what appear to be errors in Tables 6 and 7. In Table 6, column 5, lines 1 and 2 should read 104 and 425 respectively. Also, the calculated bond stress for the hooked pull-out specimens (Table 6) is based on an embedded length of 29 in., *i.e.*, the length of the hook is in-

*ACI JOURNAL, Nov. 1951, *Proc.* V. 48, p. 225. Disc. 48-17 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

†Hydro-Electric Power Commission of Ontario, Toronto, Ont., Canada, and University of Michigan, Ann Arbor, respectively.

‡Glanville, W. H., "Studies in Reinforced Concrete: Bond Resistance," *Technical Paper* No. 10, Dept. of Scientific and Industrial Research, 1930.

cluded. On the other hand, the average bond stress for hooked bars in the beam specimens is calculated ignoring the length of the hook.

Correlation between the beam and pull-out tests, as shown in Fig. 11b and 12b, is no doubt affected by the following considerations indicating the unlikelihood of longitudinal bond stress distribution in beam and pull-out specimens being identical. (1) In a pull-out specimen the concrete is in compression, but in a beam the concrete surrounding the reinforcing bar is in tension. For this reason, a pull-out specimen in which the concrete is also in tension might come closer to a beam specimen without being as expensive. (2) The force distribution in a pull-out specimen is virtually indeterminate compared to that in a beam specimen. In a beam the force in a bar at any section can be calculated from a knowledge of the applied external forces and the beam dimensions, subject to the validity of the reinforced concrete theory, provided that bond stresses and other stresses involved are below the ultimate values for the various materials. In a pull-out specimen there is no simple theory governing the bar force at any section; the force in the bar depends, among other things, on the ultimate bond stress and the bond stress-slip relationship. This is a state of affairs similar to that in statically indeterminate structures, where the load distribution depends on the stress-strain relationship of the material.

There seem to be two typographical errors on page 232; 0.2 percent offset is written as 0.002 percent and later as 0.002 in.

By ARON L. MIRSKY and LYLE K. MOULTON*

The author's method of applying SR-4 gages inside, rather than outside reinforcing bars is an interesting approach to measuring stresses inside concrete specimens. It does, however, raise several questions:

1. Do the tack welds connecting the cut portions of the bars form points of discontinuity and stress concentration? The smaller slices of the bars are fully bonded to the concrete—this being one of the main merits of the method—yet these slices are attached to the larger slices only at discrete intervals. Due to the slight difference in distance from the neutral axis of the beam, the strains in the two slices will be slightly different, resulting in a differential strain which affects the welds and hence the stress distribution in the bars. In this connection, would there be any significant change in gage readings if the finished bars were placed in the concrete beams with the joints vertical rather than horizontal?

2. What is the effect of the slots on properties of both the bars and the complete reinforced concrete section? Is the change in area-perimeter ratio of any significance? Granted that the slots are small in width and depth, and so located that their bottoms (hence, the gages) are on the calculated neutral axis of the finished bars (Fig. 1), what would have been the result of using two symmetrical slots, as did Bichara?[1]

3. What is meant by "contact surfaces" (p. 228)? Does the author mean by this that the cerese wax was spread over the entire flat surface of the smaller slice of the bar, or only on that portion lying over the wax-filled groove in the finished bar?

4. How does this method compare to the usual method of mounting gages on the surfaces of the bars, in terms of difficulty, time required, cost, and applicability to structures?

These are some of the questions which should be answered before a method of this type can be put into general experimental use. The writers do not

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know the answers; but they are interested in the best method of applying gages to reinforcing bars to be embedded in concrete.

Other similar attempts have been made, notably by Bichara,[1] who cut the bars into two equal longitudinal slices, slotted each, and attached the gages in the slots. His method of joining the two halves of the bars is not stated.

Thompson[2] used the same method, except that instead of cutting the bars in half he planed each of two bars down until only half the area of each remained, before cutting the grooves. He used tack welds to join the halves.

Wilkins[3,4] went one step further; he bored out the centers of the bars and mounted the gages on the inside surfaces by using an ingenious system of wedges and mirrors. Although this obviates discrete tack welds, only short lengths of bars can be prepared (at least initially), and special apparatus for mounting the gages is needed. However, no waterproofing is required.

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By RAYMOND C. REESE*

Professor Mains' measurements of the growth of stress along deformed reinforcing bars in the tension side of a beam show several things, the first being the fact that stress is distributed reasonably well in accordance with the theoretical bending moment curve, the second that random diagonal tension cracking causes local fluctuations in the distribution curve. Since such cracks occur more or less haphazardly, it is not practicable to write an equation that will predict where they will occur.

The most interesting fact is the localized character of stress fluctuations occasioned by cracks. This shows an excellent bonding of concrete to steel on either side of the opening. If the bars had slipped through the concrete for considerable distances, these fluctuations in the curve would have been ironed out. The fact that they are of so short a length indicates that the bond either side of the crack must have been excellent to cause such sudden changes in stress. This is a further substantiation of the value of deformations meeting ASTM A 305.

AUTHOR'S CLOSURE

In answer to the questions raised by Messrs. Mirsky and Moulton:

1. Tack welds were used on 2-in. centers to join the two parts of the split bars after tests had been made to demonstrate that this was adequate to sustain shear forces developed when

*Consulting Engineer, Toledo, Ohio.

the bar alone was loaded as a beam. Granted, there may have been some difference in strain in the two parts of the bar, but this difference must have been small or else the accumulated difference of strain over an appreciable length would have caused weld failure. Several beams were examined after test and no such weld failure was observed. In addition, the joint of the bar was placed in a vertical plane to minimize possible differences in strain.

2. Slots in the bars reduced their effective area about 15 percent, and consequently the effective modulus of elasticity by a like amount. This change in itself was probably not significant with regard to over-all beam action. However, the consequent change in area-perimeter ratio of the bars may well have improved their effectiveness in bond. It was not thought that two symmetrical slots would be enough of an advantage to justify the increased loss of area and complication of gage wiring.

3. The term "contact surfaces" was intended to denote that wax was spread over the entire flat surface of the smaller slice of the bar, as well as the flat surfaces of the larger slice. This apparently was of considerable aid in waterproofing, since the clamping and welding of the bars caused warm wax to extrude from the joint.

4. This gage technique is, in the author's opinion, strictly a laboratory operation. The difficulty, time required, and cost would prohibit its use in field structures, even if a designer could be found who would tolerate such a modified bar as a working member in his structure. This method of gage application was never intended for general experimental use, but rather for a relatively basic inquiry into the true mechanism of bond action.

It may be of interest to note that: (a) These tests were begun five years before the appearance of Bichara's note in the *Annales de L'Institut du Batiment et des Travaux Publics* in 1951. (b) This technique was described in detail to J. N. Thompson at his request after the ACI meeting of February, 1948. (c) Wilkins' study is not known to be related in any way to this work.

With regard to Messrs. Hogg's and Schenker's comment that the tests "emphasize a recognized limitation of the simple reinforced concrete theory . . . It is still true that bond and shear are related . . . ," the author wishes to comment as follows.

It is axiomatic that beam shear is the rate of change of bending moment with respect to distance along the beam axis, but only if a constant jd is assumed can bond force be a direct function of beam shear. These tests gave further evidence to demonstrate that jd is not constant, but decreases with proximity to the beam support. An attempt was made to develop a consistent relationship for this decrease of effective jd, but more specimens with a variety of dimensions and load patterns would have been required. The author would argue strongly against using these tests as a justification for continuing to consider bond to be a function of shear only.

The correction to Table 6, column 5, line 1 and 2 is gratefully accepted. It is true that bond stresses for hooked pull-out bars took account of the length of the hook. However, since nominal bond stress in a beam bar was calculated from the usual relationship of $u = V/\Sigma_o jd$, the length of hook or embedment of straight bar did not come into the picture.

Granted, a pull-out specimen with the concrete in tension would be an improvement over one with the concrete in compression. Practical proposals for such a specimen would be of interest.

Mr. Reese's comments are welcomed. They were of particular interest since he has followed this work since the first reports of early progress.

Title No. 48-18

Insulation for Protection of New Concrete in Winter*

By L. H. TUTHILL†, R. E. GLOVER†, C. H. SPENCER‡ and W. B. BIERCE‡

SYNOPSIS

Recent investigations show that new concrete with 1 percent calcium chloride, if kept from dropping below 50 F for 3 days, has sufficient protection from freezing. As an extra precaution, 3 more days protection from dropping below 32 F may be required. Since it is during the first 3 days that considerable heat is generated by the setting cement, it has been learned that insulation, within practical limits, is capable of retaining enough of this heat to keep new concrete from dropping below these temperatures. This avoids construction of special enclosures, fuel costs, and danger of fire. Examples of field applications and tests of insulation are reported together with resultant concrete temperatures. Also, there are described some aids which have been developed for determining what insulation is necessary under various conditions.

NEW REQUIREMENTS FOR PROTECTION

For some time it has been suspected that, with the added maturity imparted by the use of 1 percent of calcium chloride by weight of the cement in concrete in freezing weather, considerably fewer than the 14 days' protection customary with many constructing agencies were sufficient for protection of new concrete from freezing. The excellent strength and durability of many concrete structures known to have had relatively superficial protection indicated this. Consequently, when the concrete on one job inadvertently received considerably less than the amount of protection usually required, drill cores were tested and found to have the proper strength and durability. Concurrently, the program of tests of concrete cured at low temperatures recently reported§ was performed. The results of these tests are summarized in Fig. 1.

Several months ago, as the results emerged from these investigations it became apparent the period of protection formerly specified could be shortened safely. This former requirement was that the concrete be placed at a temperature of 50 F or above, not be permitted to fall below that temperature for the first 3 days, and be kept from freezing for 11 days thereafter. The new specification is exactly the same for the first 3 days, but the protection

*Received by the Institute July 23, 1951. Scheduled to be presented at the ACI 48th Annual Convention, Cincinnati, Ohio, Feb. 26-28, 1952. Title No. 48-18 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 3, Nov. 1951, *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

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‡Construction Engineers, Hungry Horse and Fort Sumner Projects, Bureau of Reclamation.

§Shideler, J. J., Brewer, H. W. and Chamberlin, W. H.; "Entrained Air Simplifies Winter Curing," ACI JOURNAL, Feb. 1951, *Proc.* V. 47, p. 449.

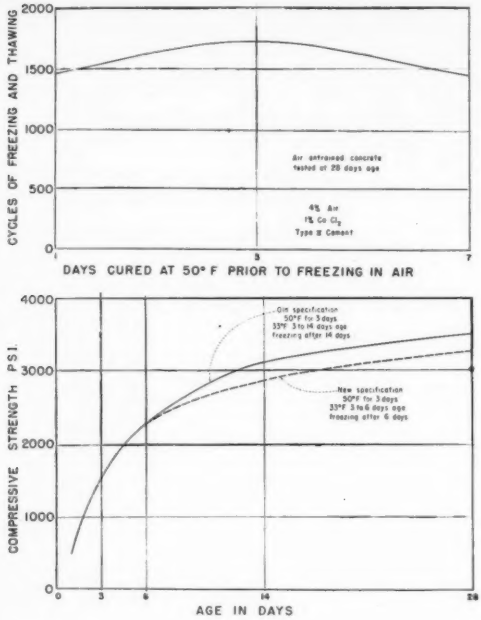


Fig. 1—Effect of curing temperature on strength and durability of concrete containing 1 percent of calcium chloride by weight of cement

Three days protection at 50 F gives maximum durability

Shorter protection reduces 28-day strength only 7 percent; ultimate strengths are equal

from freezing thereafter for 11 days has been eliminated except that instead, 3 days of protection from freezing is required as an added precaution for new concrete placed in the fall and winter which must face considerable repeatedly freezing weather before it has the opportunity for further strength development in mild, and probably damp, spring weather. Beyond this it is required only that the protection be discontinued in such a manner that the drop in temperature of any part of the concrete will be gradual and not exceed 40 F in 24 hours.

OPPORTUNITY FOR INSULATION

Insulation for the protection of winter concrete work from freezing is not new.* Straw has been commonly used, especially on flat work. But to a large extent, the long period of protection heretofore required has necessitated the use of steam or otherwise heated enclosures for the protection of formed concrete.

When it was concluded that the period the concrete must be kept from freezing could be materially reduced, it appeared that here was a new and wider opportunity for the use of insulation. During the first 3 days after concrete is placed, considerable heat is generated by the setting processes. Here was a source of heat which, if sufficiently retained, might be sufficient

*See Portland Cement Assn. information sheet No. ST-21, "Concreting in Cold Weather."

to keep the concrete from dropping below 50 F for the first 3 days if only this protection was necessary, and would keep it from dropping below freezing for the next 3 days when that additional protection was required. Many expensive and sometimes disastrous fires have resulted from combustion heating of enclosures around new concrete. Considerable labor for building and moving enclosures, and fuel for heating, must be expended with even the shortened protection requirements, unless the heat generating in the hardening concrete could be confined to serve this purpose. With the prospect of these advantages, computations from known and assumed data were made which indicated that insulation within practical limits would be capable of providing the required protection for the shorter interval now considered sufficient for concrete containing 1 percent calcium chloride by weight of the cement. In two cases which will be described, construction contractors found the advantages of insulation sufficiently attractive to give it a full-scale trial under job conditions.

WINTER CURING

It should be mentioned, however, that neither insulation nor the shortened period of protection can be used where water curing for a longer period is required. On Bureau of Reclamation work this is usually avoided because of extensive use of a white-pigmented sealing compound for curing which can be applied, after proper wetting of the concrete, during the first time the weather is above freezing after the insulation is removed. Actually the water curing needed in winter is primarily to offset the drying occurring in combustion-heated enclosures. Concrete enclosed in insulated forms or other effective insulation will rarely lose enough moisture at 50 F and lower to impair curing. For mass concrete which will be exposed to subsequent wetting from rains and snows, we have concluded that water curing in the winter is unnecessary after the period of protection. Some practitioners will feel that it is unnecessary for any concrete under these conditions.

FIELD TRIAL OF INSULATED FORMS AT HUNGRY HORSE DAM

Fig. 2 and 3 illustrate how the steel form panels for the face of the dam were insulated. A 1 x 2-in. wooden strip was fastened to the outside edges of the horizontal steel ribs of the form panels. To this, reinforced, waterproof Sisalkraft paper covered with chicken wire is attached as the interior space is filled with planer shavings. The wooden strip also tends to block the rapid heat transfer through the steel ribs.

Fig. 4 shows the temperature history of concrete immediately inside this form. Although the concrete was placed at only 45 F instead of 50 F as desired, the temperatures of surface concrete reached 50 F within the next 24 hours and remained there for about the next 3 days, despite a sharp drop in the air temperature. Two months later, in January, it was above freezing and 13 F above the mean air temperature. It was possible to obtain the continuing data of Fig. 4 because the forms were left in place on the last lift

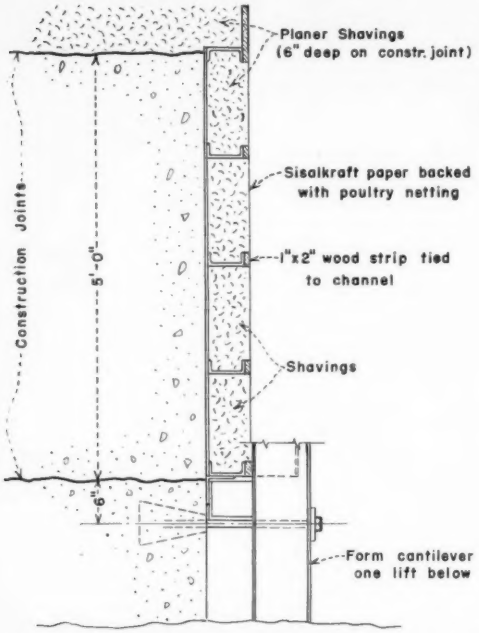


Fig. 2—Detail of insulation of steel forms for downstream face of Hungry Horse Dam

Fig. 3—Filling forms shown in Fig. 2 with planer shavings

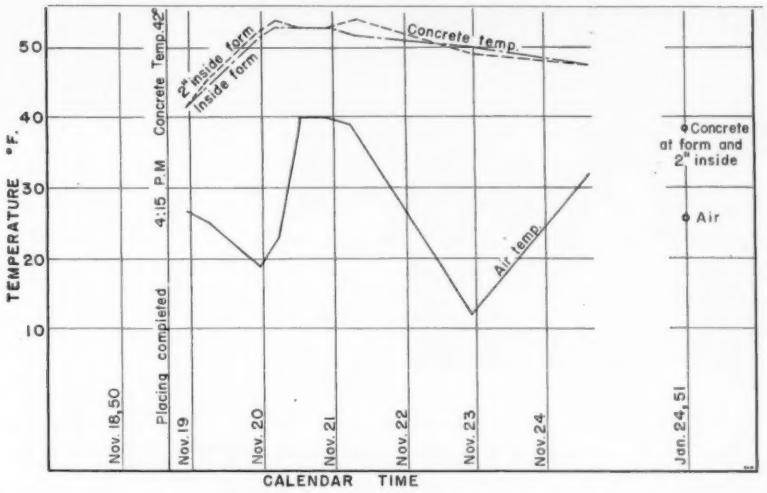


Fig. 4—Temperature of concrete resulting from insulating downstream steel forms of Hungry Horse Dam with wood shavings

of concrete it was found practical to place, because of increasingly cold weather. With increasing severity of the cold, cleanup, aggregate processing, and other construction operations became so difficult that on their account concrete work was discontinued; it was not because of difficulty in keeping the concrete up to the required temperatures.

Originally, it was planned to leave the forms in place at least 3 days, then as they were moved up, replace them by wood frame panels covered with Sisalkraft paper and held tightly against the concrete face so as to afford a 1-in. air space. This would not only tend to conserve the heat from the concrete and to keep the temperatures above freezing for another few days, but it would greatly slow the rate at which the surface temperature would drop. Actually these were not used because severe cold came on so quickly that the work was closed down till spring; and the forms were left in place.

Horizontal and vertical joint surfaces were supplied with heat from large Evans kerosene-burning orchard heaters with downward reflectors. It was expected that edges at the perimeters of the horizontal surfaces would require additional protection, and it was planned to use the air space panels described above. However, as the shutdown came so quickly, and as it was not expected concrete work would be resumed until spring, 6 in. of planer shavings were used to cover these surfaces. This was not only to keep concrete up to proper curing temperatures initially, but to keep them from dropping later to temperatures low enough to cause transverse cracking in the long, narrow block surfaces.

No insulated forms were used in the powerhouse before work was discon-

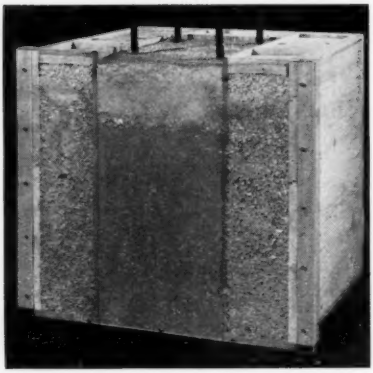


Fig. 5—Mock up of wooden forms insulated with 6 in. of planer shavings. During a mean temperature of 20 F, this insulation is sufficient to keep the corners up to 50 F for the first 3 days

tinued for the winter. For the cold nights only, prior to the shutdown, the wooden forms and several inches of shavings or sawdust on the concrete at the top of a wall were sufficient protection for the heavy walls. It was computed, however, that in a 12-in. wall of concrete containing 5½ sacks of cement per cu yd, insulated as shown in Fig. 5, both sides and ends with planer shavings in forms with 6-in. studs and sheeted outside the studs, and with 6 in. of shavings on top, protection as specified would be obtained for the *edges* at mean temperatures down to 0 F; the *corners* at mean temperatures down to 20 F. At a mean temperature of 20 F, the *edges* would be 60 F at 72 hours, 50 F at 6 days; the *corners* would be 50 F at 72 hours, and 40 F at 6 days.

Edges and corners are, of course, more vulnerable to loss of temperature and freezing than the faces, hence, if minimum temperatures are held at edges and corners, ample temperatures will be maintained elsewhere.

FIELD TRIAL OF INSULATING MATS FOR CONCRETE CANAL LINING

At the Fort Sumner Project in New Mexico, about 4000 ft above mean sea level, winters are mild enough to continue daytime concrete work most days, but the new concrete is definitely in danger of damage from freezing at night. The contractor for placing 3-in. concrete lining in the 24-ft perimeter Main Canal asked for ideas on simple means of protection. Open salamanders were ineffective and it was economically out of the question to house such work. Insulating mats were suggested, at least for a trial. The contractor forthwith obtained 30 blankets, 10 x 24 ft, of 1-in. spun glass bound on both sides by waterproof, reinforced Sisalkraft paper. He placed the lining by hand in alternate panels 10 ft wide, averaging about 10 panels a day. Thus, each blanket completely covered each 10-ft panel where it remained for 3 days except when unusually cold weather stopped the work, and they were then left until needed when the work resumed. Pieces of 2 x 4-in. lumber

Fig. 6—Canal lining concrete was kept well up to required temperatures during freezing nights with these 1-inch blankets of spun glass. See Table 1

were arranged as shown in Fig. 6 to hold the edges of the mats in place. As soon as the mats were removed, white-pigmented sealing compound was applied for further curing.

These insulating mats were not only successful in keeping temperatures closely up to requirements, as indicated in Table 1, but evidently they were an economical solution of the contractor's problem, for he ordered an additional 15 mats. They were reused many times and were in fair condition after being used approximately 20 times. Moisture and rough handling shorten their life; bound edges would greatly improve the strength and life of the blankets by preventing tearing of the covers and filler. The new mats were ordered 11 ft wide to provide better protection at the edges of the concrete.

Obviously, the success of insulation for protection of thin layers of concrete on a subgrade depends on the earth being not only free of frost but also several degrees above freezing. Day temperatures were above freezing and salamanders were used in the excavation to prevent frost. Actually, excavation removed the coldest earth and two tests of subgrade temperatures, Dec. 28, 1950 (Table 2), showed that there was enough heat in the subgrade to support insulation of the thin slab from low air temperatures.

LABORATORY TESTS OF INSULATED FORM

As a further check on the assumptions and factors used in computing the effect of various insulating covers under certain winter conditions, a laboratory

TABLE 1—CONCRETE AND AIR TEMPERATURES

Date	Age	Concrete temperature, F		Air temperature, F	
		Max.	Min.	Max.	Min.
1-27-51	Placed	58	48	50	27
1-28-51	1 day	54	40	56	12
1-29-51	2 days	48	45	22	6
1-30-51	3 days	48	44	26	8
1-31-51	4 days	33	31	16	−15

TABLE 2—SOIL TEMPERATURES

Time	8:45 am	11:30 am
Air temperature, F	17	32
Earth temperature, 0.3 ft down, F	32	38
Earth temperature, 1.0 ft down, F	31	34
Earth temperature, 2.0 ft down, F	42	40
Earth temperature, 3.0 ft down, F	44	44
Depth of excavation, ft	7	6
Weeks excavated before test	2	4
Material	Earth and shattered rock	Sandy loam

test of temperature results inside an insulated form was made concurrently with the foregoing field uses of insulation. Fig. 7 shows how a 2-ft cube of 6-sack concrete was surrounded on all sides by a plywood form insulated with 3½ in. of dry shavings. Ten thermocouples were embedded so as to record the temperature of the concrete cube at its center, five faces, and four corners. A Brown Electronic Recorder made a continuous record of these temperatures for 7 days. The form was chilled to an initial temperature of 20 F. The concrete was mixed and placed at a temperature of 55 F. Fig. 8, which shows the average recorded temperatures of center, faces, and corners, shows that all of this concrete, including the vulnerable corners, remained above 50 F for 3 days and above freezing for 7 days, a completely adequate protection for concrete containing 1 percent of calcium chloride by weight of the cement.

CALCULATION OF CONCRETE TEMPERATURES

Where plans are made beforehand to use insulation for the protection of concrete to be placed in cold weather, some means of estimating the temperatures which will be maintained in the concrete under the anticipated weather conditions are needed. The graphs of Fig. 9, 10, and 11 have been prepared

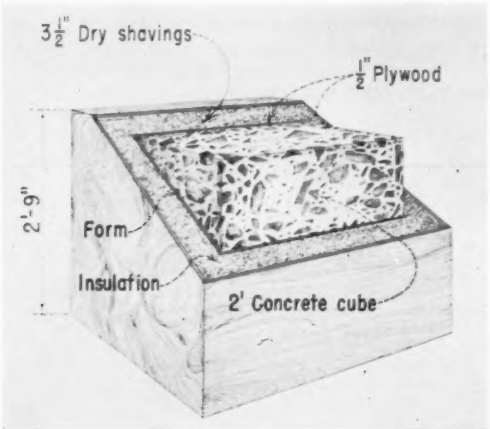


Fig. 7—Cutaway section showing construction of 2-ft block and insulated form tested in laboratory for temperature control. See Fig. 8

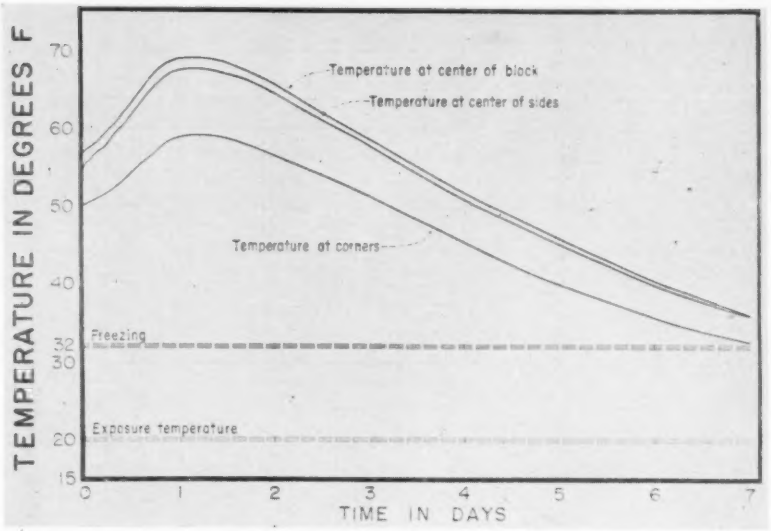


Fig. 8—Insulated form keeps corners of a 2-ft cube above 50 F for 3 days and above freezing for 7 days. Placing temperature 55 F, air temperature 20 F, cement content 6 sacks per cu yd, form insulation 3½ in. dry shavings. See Fig. 7

to supply this need. Before proceeding to a description of these graphs, it will be advisable to explain some of the reasons why the forms shown were chosen.

The problem of computing temperatures in a solid body involves an unusually large number of variables. These are three dimensions, three coordinate positions, initial, external, and transient concrete temperatures, time, three thermal properties of the concrete, the surface emissivity and, in our case, heat generation within the body. Although mathematical expressions can be obtained which account for nearly all these factors, they are too cumbersome to be attractive for computation purposes. If an attempt is made to graph these expressions, it will be found that the number of variables is so large that none of the common forms for presenting data are adequate. These difficulties have been overcome in the present case by use of a *product law.* By using this device, graphs for the one-dimensional case can be used for two- and three-dimensional cases by multiplying together values obtained from the graphs. In this way, the number of variables can be reduced so that charts can be prepared provided that the other variables are combined in certain ways. These combinations will be identified in the following descriptions.

Fig. 9 is a graph prepared for a wall of thickness L exposed on both sides. The notation used is as follows:

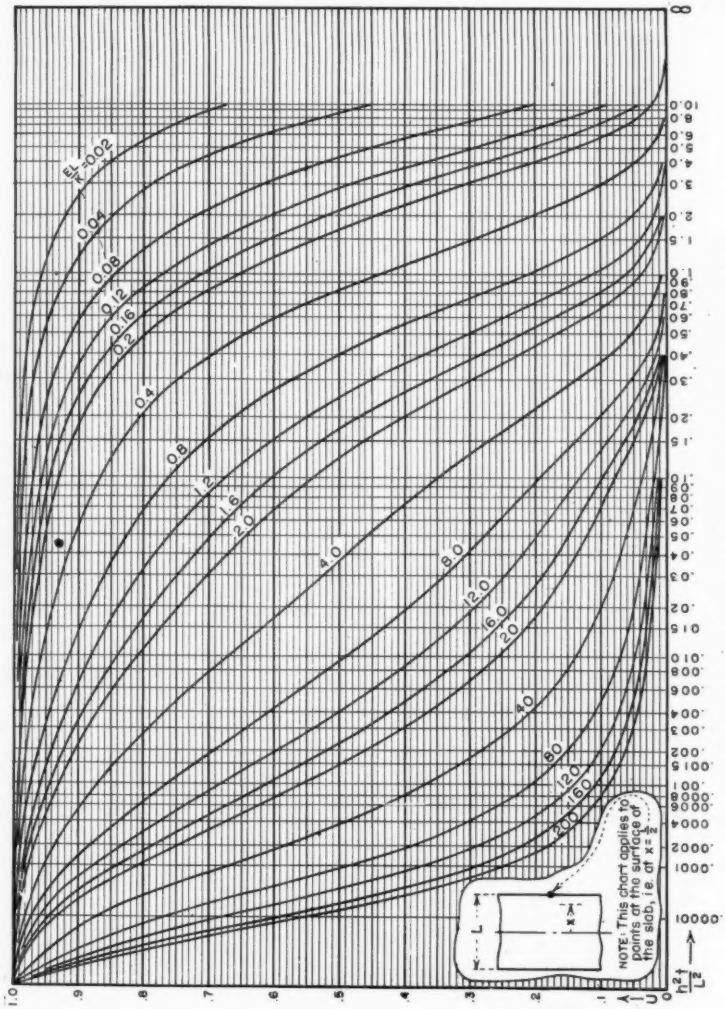


Fig. 9—Plot of U versus h^2t/L^2 for points at the surface of a slab

C = Specific heat of the concrete (Btu per lb per deg F)

E = Surface emissivity. This figure represents the heat loss per unit of area per unit of time per unit of temperature differential between the surface of the concrete and its surroundings (Btu per sq ft per hour per deg F)

$h^2 = \frac{K}{C\rho}$ = "Diffusion constant" for the concrete (sq ft per hour)

K = Conductivity of concrete (Btu per ft per hour per deg F)

L = Thickness of the wall (ft)

t = Time (hours)

$U = \frac{\Theta}{\Theta_o}$ = Ratio of the temperature differential at time t at the point indicated to the original temperature differential. These differentials are the differences between the temperatures at the points indicated and the external temperature. (Dimensionless)

Θ_o = Temperature differential at time zero. It is assumed that the temperature is uniform throughout the mass at this time which will, in many cases, be the time of placing the concrete (deg F)

Θ = Temperature differential at the time t (deg F)

ρ = The density of the concrete (lb per cu ft)

In Fig. 9 the variables have been grouped into the combinations h^2t/L^2, EL/K and Θ/Θ_o or U. If values of the first two of these are at hand for a specific case, we may start on the abscissa with the value h^2t/L^2, follow up the vertical lines until the curve for EL/K is reached, and then go to the left to read U on the ordinate. The value of U so obtained represents the ratio Θ/Θ_o, where, in this case, the temperature Θ is surface temperature at time t. Fig. 10 is similar except that the temperature differential Θ is at the center of the slab. Fig. 11 gives, in a somewhat similar way, the temperatures at the surface of a mass of great depth. In each case, the solution represents the decay of an initially existing temperature differential toward an external exposure temperature. Temperature differentials are, of course, computed from the relation $\Theta = U\Theta_o$.

The extension to a prismatic body is made by use of the *product law* in the following way. Suppose the body has the dimensions L_1 and L_2. Use the two values of L in succession to get corresponding values of U. Call these U_1 and U_2. If the U values were obtained from Fig. 9, the product U_1U_2 represents the U value for an *edge*. If U_1 came from Fig. 9 and U_2 came from Fig. 10, then the product U_1U_2 represents the U value for the middle of the side whose dimension is L_2. If U_1 came from Fig. 10 and U_2 came from Fig. 9, then the product U_1U_2 gives the U value for the middle of the side whose dimension is L_1. If both come from Fig. 10, then the product U_1U_2 is the U value for the center of the prism.

In case the body has the form of a rectangular parallelepiped with dimensions L_1, L_2, and L_3 and each length is used in turn to obtain a U value, then the U value for a *corner* is obtained by forming the product $U_1U_2U_3$ where all the U values are obtained from Fig. 9. If two U values came from Fig. 9 and one comes from Fig. 10, the product $U_1U_2U_3$ gives the U value for the mid-point of an edge. If one U value comes from Fig. 9 and two come from Fig. 10, the product $U_1U_2U_3$ gives the U value for the middle of a side. If

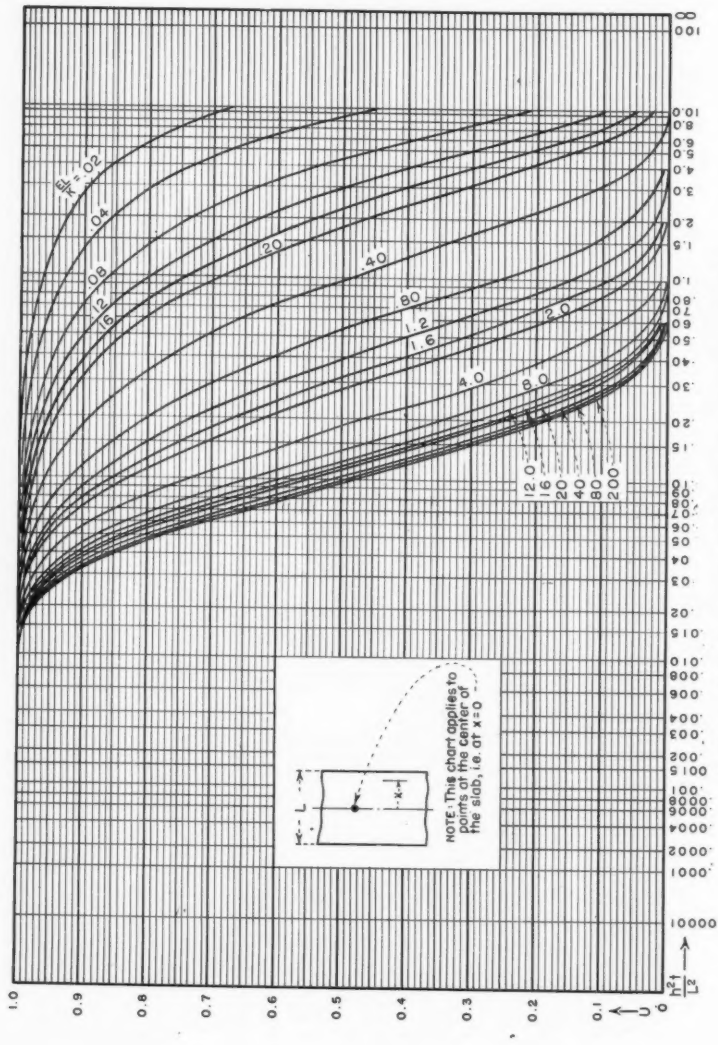


Fig. 10—Plot of U versus h^2t/L^2 for points at the center of a slab

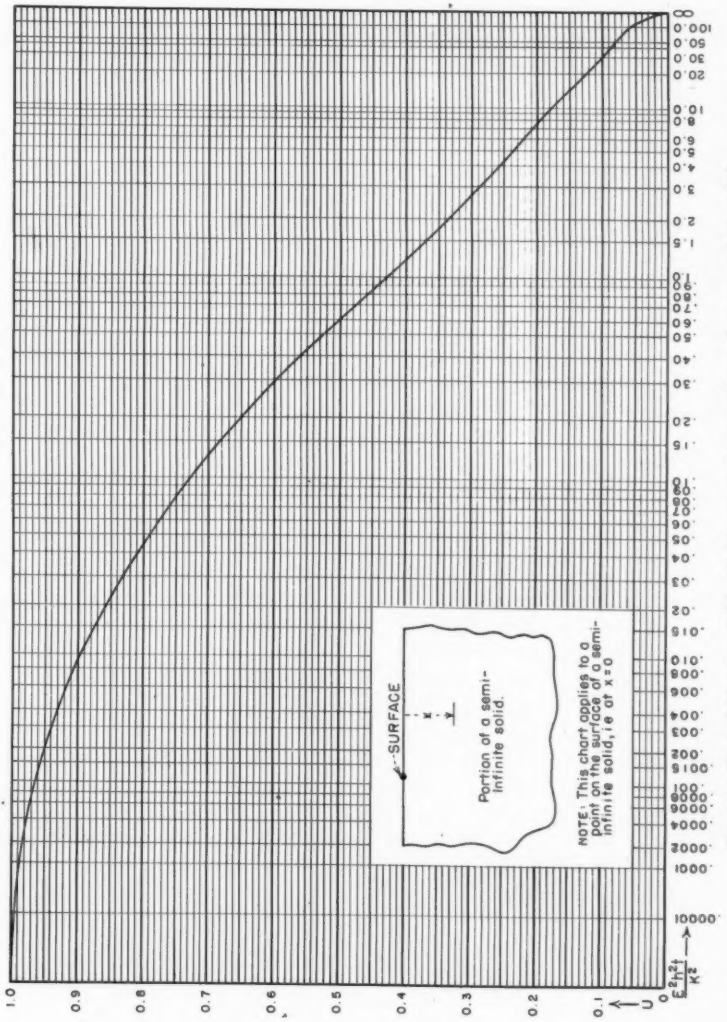

SURFACE
Portion of a semi-infinite solid.
NOTE: This chart applies to a point on the surface of a semi-infinite solid, i.e. at x = 0
U
$\frac{E^2h^2t}{K^2}$

Fig. 11—Plot of U versus E^2h^2t/K^2 for points at the surface of a semi-infinite solid

all three U values come from Fig. 10, the U value obtained applies to the center of the solid. These relations are all exact and remain so even if the E values corresponding to the dimensions L_1, L_2, and L_3 are all different.

These processes relate only to the decay of an initial temperature toward an assumed constantly maintained external temperature. How, then, is one to deal with the effect of heat generation and of changing external temperatures, such as might be provided, for example, by a cold wave? The answer is that these solutions obey the law of superposition. The effect of heat generation is accounted for by substituting a stepped curve for the smooth curve of temperature rise obtained from an adiabatic calorimeter test on the concrete, applying the procedure described to each step and adding the results. This is not as formidable an undertaking as it sounds because the steps may be made larger for the longer time periods. Thus, if temperatures are to be computed at the end of 24 hours, 4-hour increments may be short enough. If the computation is to be made at the end of a week, then 1-day increments should give a close enough approximation for engineering purposes. This process is approximate, of course, and the greater the number of increments taken, the nearer will the result approach analytical perfection.

Temperature changes are taken care of in much the same way. The principle of assuming that a temperature differential, once applied, remains forever is followed. Thus, a cold wave lasting for 3 days would be computed by applying the appropriate differential at the time the cold wave begins. At the end of the cold period, the original differential is not removed, but the external temperature is restored by adding a new differential equal in magnitude, but opposite in sign, from the first. It may be noted that in all cases the temperatures computed refer to the last external temperature change applied, but that in computing the Θ value from the relation $\Theta = U\Theta_o$, the Θ_o values remain unchanged. Fig. 11 is used where the thicknesses are so great that Fig. 9 cannot be used. The insulation applied affects the E values.

Experience with these methods has shown that accurate data must be used if close correlation is to be obtained. The reason is that, if the insulation is skillfully chosen, the temperature rise due to heat generation will nearly offset the natural cooling. Thus, at the end of 3 days each of these factors may be about 30 F but the net temperature change may be near zero. It is easy to see, therefore, how an error in computing one of the factors can make a relatively large error in the final result. This possibility should be kept in mind when using these methods.

Example 1

A computation of temperatures in the 2-ft cubical test block of Fig. 8 illustrates the use of the charts and of the product law. Temperatures at the corners, at the mid-point of the sides, and at the center of the block, 3 days after placing, will be computed. The following data are required:

Thermal properties of concrete:

Specific heat	C = 0.230 Btu per lb per deg F
Conductivity	K = 1.56 Btu per ft per hour per deg F

Density	ρ	= 152.9 lb per cu ft
Diffusivity	h^2	= $K/C\,\rho$ = 0.0444 sq ft per hour
Placing temperature		55 F
External temperature		20 F
Emissivity		0.15 Btu per sq ft per hour per deg F

The detail of the computations is as follows: Column 1 (Table 3) contains the time after placing, while column 2 contains the temperature rise which would be produced in the specimen if the setting heat were not allowed to escape. Column 3 contains the differences of the quantities in column 2. These differences represent the adiabatic rise in each 6-hour interval. These changes can be assumed to occur at either the beginning or end of the interval. In this case they have been assigned to the beginning. The time interval should be chosen short enough so that it will not make much difference which choice is made. Column 4 contains the lengths of time each of the corresponding increments is assumed to have been in existence at the end of the 3-day period. The values in column 5 are computed from the figures in column 4 and the U values for the surface and the center of the cube are found from Fig. 9 and 10 and entered in columns 6 and 7, respectively. The values for U_2 and U_3 are the same because the lengths and insulation are the same. Columns 8, 9, and 10 contain products of U values from columns 6 and 7. These three columns contain the appropriate U values for a corner, the middle of a side, and the center of the cube, respectively. Columns 11, 12, and 13 contain the products of the increments in column 3 with the U values in columns 8, 9, and 10, respectively. The sums of the quantities in columns 11, 12, and 13 represent the temperature rise above the placing temperature, produced by setting heat alone, at the end of the 3-day period.

We have yet to account for the effect of the 35 F temperature differential between the placing temperature and the external temperature. This dif-

TABLE 3—COMPUTATION OF TEMPERATURES IN 2-FT CUBE TEST BLOCK

1	2	3	4	5	6	7	8	9	10	11	12	13
Time, hr	Temp. rise, F	Increments, F	Life at 72 hr	$\frac{h^2t}{L^2}$	U_1	$U_2 = U_3$	$U_1U_1U_1$	$U_1U_2U_2$	$U_2U_3U_3$	°F $U_1U_1U_1$	°F $U_1U_2U_2$	°F $U_2U_3U_3$
0	0	10.2	72	0.799	.720	.755	.373	.410	.430	3.80	4.18	4.39
6	10.2	8.5	66	0.733	.735	.778	.397	.445	.471	3.37	3.78	4.00
12	18.7	4.8	60	0.666	.750	.792	.422	.470	.497	2.02	2.26	2.39
18	23.5	3.5	54	0.599	.774	.815	.464	.514	.541	1.62	1.80	1.89
24	27.0	3.1	48	0.532	.796	.835	.504	.555	.582	1.56	1.72	1.80
30	30.1	2.6	42	0.466	.812	.854	.535	.592	.623	1.39	1.54	1.62
36	32.7	2.4	36	0.400	.835	.878	.582	.644	.677	1.40	1.54	1.62
42	35.1	2.2	30	0.333	.855	.900	.625	.692	.729	1.38	1.52	1.60
48	37.3	2.0	24	0.266	.875	.922	.670	.744	.784	1.34	1.49	1.57
54	39.3	1.8	18	0.200	.898	.943	.724	.798	.838	1.30	1.44	1.51
60	41.1	1.7	12	0.133	.917	.964	.771	.852	.896	1.31	1.45	1.52
66	42.8	1.7	6	0.067	.948	.990	.899	.929	.970	1.53	1.58	1.65
72	44.5	—	0	0	1.000	1.000	1.000	1.000	1.000	—	—	—
										22.02	24.30	25.56

Placing temperature = 55 F External temperature = 20 F
Temperature at corner = (55-20) $U_1U_1U_1$ + 22.02 + 20 = 55.08
Temperature at middle of side = (55-20) $U_1U_2U_2$ + 24.30 + 20 = 58.65
Temperature at center of block = (55-20) $U_2U_3U_3$ + 25.56 + 20 = 60.61
EL/K = (0.15) (2) / 1.56 = 0.192
h^2/L^2 = 0.0444/4 = 0.0111

ferential has existed for the entire 72-hour period and its effect may be computed by using the U values at the top of columns 8, 9, and 10 as appropriate. For the corner, the appropriate value comes from column 8. This computation then proceeds as follows:

External temperature	20 F
Parts remaining of the 35 F difference between the placing temperature and the external temperature 35 $U_1U_1U_1$ = (35) (0.373) =	13.0
Parts remaining of the setting heat increments, from column 11	22.0
Total	55.0 F

This is the computed temperature at the corners at the end of 3 days. The corresponding computations for the middle of a side and for the center of the cube are shown at the bottom of Table 3. The computations can be referred to the placing temperature by multiplying the external temperature differential by $(1-U)$ thus:

Placing temperature	+ 55 F
− 35 (1 − 0.373) =	− 21.9
Setting heat	+ 22.0
Total	55.1 F

When the computation is made in this way, it brings out the point that, in this case, the two factors of loss of heat through the insulation and heat gain from chemical action counterbalance each other within 0.1 F while each factor is about 22 F. The results of the computations are therefore obtained as a small difference between relatively large numbers and the outcome of the computation becomes sensitive to variations in the assumed data. In this case, for example, the use of $E = 0.10$ instead of $E = 0.15$ will increase the computed temperature 10 F.

Since a close balance between heat loss and heat gain will be the result of a skillfully chosen insulation in any field application, the importance of avoiding an overestimate of the effectiveness of the insulation will be appreciated. Another factor which will bear watching is the data on adiabatic heat rise. Such data are often obtained from an adiabatic calorimeter test started at 70 F. The speed of the chemical reaction is, however, strongly influenced by temperature* and, therefore, care should be taken to use data appropriate for the temperature level anticipated.

This difficulty is inherent and it is therefore advisable to check the computations against field performance at every opportunity to obtain data for correcting the assumptions used.

Canal linings

Some modification of the computation procedures described previously must be made in the case of canal linings because the thermal properties of the soils on which they are placed exert such a profound influence. The heat-storage capacity of the lining will generally be small compared to that of the earth beneath it, and it will, therefore, be permissible, as an approximation, to assimilate the lining to an equivalent thickness of soil and to treat the combi-

*See, for example, Fig. 3-1 and 3-2, pp. 792 and 793, "Long Time Study of Cement Performance in Concrete," by William Lerch and C. L. Ford, ACI JOURNAL, Apr. 1948.

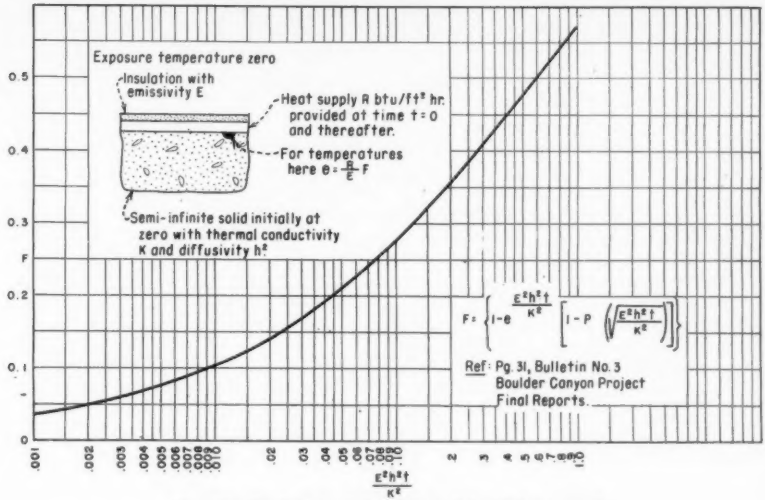

Exposure temperature zero
Insulation with emissivity E
Heat supply R btu/ft² hr. provided at time t = 0 and thereafter.
For temperatures here θ = R/E F
Semi-infinite solid initially at zero with thermal conductivity K and diffusivity h²
F
Ref: Pg. 31, Bulletin No. 3 Boulder Canyon Project Final Reports.

Fig. 12—Radiating semi-infinite solid with heat supply

nation of lining and soil as a semi-infinite solid of soil. Fig. 11 will serve for computation of the effects of external temperature changes as well as before but the computation for the effects of heat generation and of initial temperatures of the concrete must be modified because the soil is inert. The semi-infinite solid idealization can be extended to cover these effects also if it is assumed that the heat coming from a placing temperature above that of the surroundings and from chemical action is applied to the surface of the solid beneath the insulation. Fig. 12 was prepared to assist this part of the computation. It was plotted from formulas derived* for the case of a semi-infinite solid which radiates to a temperature zero but which receives at the same time a heat supply of R Btu per hour per square foot at its surface. The ultimate temperature rise in such a solid will be R/E. Whereas, before, the effect of setting heat was accounted for by a stepwise approximation based on increments of temperature rise, these effects in the present case will be accounted for by a similar stepwise computation based on rates. These rates are connected with the temperature rise increments used previously in a manner which will be made clear in the following example. The purpose of the computation is to obtain temperature values at the surface of the concrete.

Example 2

As an example of the use of the charts for estimating the temperature of a canal lining, the temperature of the concrete at age 3 days will be computed for the conditions described in Table 1. The required data are:

*See Par. 23 and 24, *Bulletin* 3, Part VII of the Boulder Canyon Project Final Reports, "Cooling of Concrete Dams."

Placing temperature of concrete (assumed) 50 F
Soil surface temperature at time of placing (assumed) 32 F
Thickness of concrete lining 0.25 ft
Insulation: Rockwool mat between Sisalkraft paper facings.
(Conductance 0.25 Btu per sq ft per hr per deg F
Cement content of concrete: 4 sacks per cu yd
Calcium chloride content: 1 percent of cement by weight

Thermal properties of concrete (assumed):

Specific heat	C_c	= 0.230 Btu per lb per deg F
Conductivity	K_c	= 1.56 Btu per ft per hour per deg F
Density	ρ_c	= 153 lb per cu ft
Diffusivity	$K_c/C_c\rho_c$	= 0.0444 sq ft per hour

Thermal properties of soil (assumed):

Specific heat	C	= 0.45
Conductivity	K	= 0.90 Btu per ft per hour per deg F
Density	ρ	= 100 lb per cu ft
Diffusivity	$K/C\rho$	= 0.020 sq ft per hour
Emissivity, with insulating blanket	E	= 0.25 Btu per sq ft per hour per deg F

We will take account of the placing temperature, the soil temperature, the setting heat, the temperature gradient in the ground, and the effect of the external air temperature changes. As stated previously, the temperatures will be computed on the basis that they occur in soil. The thermal properties of concrete will therefore find use only for making estimates of the heat contributed by the concrete. As before, a 6-hour time increment will be chosen. On this basis the rate of heat supply to account for the $(50 - 32) = 18$ F difference between the concrete and ground temperatures at the time of placing will be computed as $C_c \rho_c \times 0.25 \times 18/6 = 26.4$ Btu per sq ft per hr. And this rate will be assumed to be present during the first 6 hours.

The rate of heat supply due to a ground temperature gradient of 4 F per foot is $(4.0)K = 3.60$ Btu per sq ft per hour. The detail of this computation is shown in Table 4. The first column shows the number of hours after placing the lining. The "adiabatic temperature rise" in column 2 is the temperature increase which the concrete would show if cured in an adiabatic calorimeter. These quantities are greatly influenced by the temperature*. Column 3 contains the differences of the quantities in column 2, and column 4 contains the equivalent heat supply computed from the entries in column 3. The formula used is shown at the bottom of the tabulation. The rate from initial temperature difference shown in column 5 is sufficient to accumulate the initial temperature excess of concrete over soil in the first 6-hour time interval. The purpose of this conversion is to permit computation of the effect of placing temperatures by use of Fig. 12. The rate from ground gradient in column 6 accounts for the heat coming up through the ground. In the winter this gradient averages about 4 F per ft at the ground surface and the heat brought upward amounts to about 3.6 Btu per sq ft per hour. Column 7 contains the

*Lerch, William and Ford, C. L., "Long-Time Study of Cement Performance in Concrete, Chapter 3—Chemical and Physical Tests of the Cements," ACI JOURNAL, Apr. 1948, *Proc.* V. 44, p. 745.

TABLE 4—COMPUTATION OF CANAL LINING TEMPERATURES 3 DAYS AFTER PLACING

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Time, hr	Assumed adiabatic temp. rise, °F	Increment of rise, °F	Rate* from setting heat	Rate* from initial temp. diff.	Rate from ground gradient	Total rate, R, Btu per sq ft per hr	R	Life of increment at 72 hr	$\frac{E^2h^2t}{K^2}$	F†	$\frac{F-\Delta R}{E}$	Mean daily air temp., F	Differences,‡ T	Life at 72 hr	$\frac{E^2h^2t}{K^2}$	U§	TU
0	0	4.4	6.45	26.4	3.60	36.45	+36.45	72	0.1110	0.286	+41.70	38.5	+6.5	72	0.1110	0.720	+4.7
6	4.4	3.9	5.72	0	3.60	9.32	−27.13	66	0.1018	0.278	−30.17	38.5					
12	8.3	3.5	5.13	0	3.60	8.73	−0.59	60	0.0926	0.267	−0.63	38.5	+4.5	60	0.0926	0.735	+3.3
												34.0					
18	11.8	3.1	4.54	0	3.60	8.14	−0.59	54	0.0833	0.256	−0.60	34.0					
24	14.9	2.7	3.96	0	3.60	7.56	−0.58	48	0.0741	0.245	−0.57	34.0					
30	17.6	2.5	3.66	0	3.60	7.26	−0.30	42	0.0648	0.231	−0.28	34.0					
36	20.1	2.2	3.22	0	3.60	6.82	−0.44	36	0.0555	0.218	−0.38	34.0	+20.0	36	0.0555	0.780	+15.6
												14.0					
42	22.3	2.0	2.93	0	3.60	6.53	−0.29	30	0.0463	0.201	−0.23	14.0					
48	24.3	1.8	2.64	0	3.60	6.24	−0.29	24	0.0370	0.185	−0.21	14.0					
54	26.1	1.6	2.34	0	3.60	5.94	−0.30	18	0.0278	0.162	−0.19	14.0					
60	27.7	1.5	2.20	0	3.60	5.80	−0.14	12	0.0185	0.136	−0.08	14.0	−3.0	12	0.0185	0.860	−2.6
												17.0					
66	29.2	1.3	1.91	0	3.60	5.51	−0.29	6	0.0093	0.100	−0.12	17.0					
72	30.5							—	—	—	—	17.0					
											+8.24						+21.0

Computed temperature of the surface of the concrete at 72 hours after placing = 17.0 + 8.24 + 21.0 = 46.2 F.

*Rate = $C_c \rho_c (0.25)$ °F/6

†From Fig. 12

‡The first difference is with respect to the 32 F assumed ground temperature.

§From Fig. 11.

totals of the items in columns 4, 5, and 6. Column 8 contains the differences of the figures in column 7. The first figure represents the starting rate and is plus because the factors of columns 4, 5, and 6 all tend to cause a temperature rise. The succeeding figures are negative because they indicate a decrease in rate. Column 9 shows the time the items in column 8 have been in existence at the end of the 3-day period. The quantities in column 10 are computed from the items in column 9 and used to read the items in column 11 from Fig. 12. The figures in column 12 are computed from the items in columns 8 and 11 and represent the temperature changes produced at the end of the 3-day (72-hour) period by the heat supply rates of column 7. Column 13 is obtained by averaging the figures in Table 1. The concrete is assumed to have been placed at noon and the mean daily air temperature to prevail over the whole 24-hour period of each day. Column 14 contains the differences of the quantities of column 13. The first difference is taken with respect to the assumed ground temperature. The remaining differences are plus if the temperature is dropping. Columns 15 and 16 are similar to columns 9 and 10. The figures in column 17 are obtained from those of column 16 by use of Fig. 11. Column 18 contains the products of figures in columns 14 and 17. Figures in column 18 represent the parts remaining of the differences of column 14 at the end of the 3-day period. Thus, of the 20.0 F differential occurring because of a drop of air temperature 36 hours after placement, 16.0 F remains at the end of the 3-day period. In other words, the 20 F temperature drop at 36 hours only succeeds in causing a temperature drop of $20 - 15.6 = 4.4$ F in the concrete at the end of the 3-day period. The computed concrete temperature at the end of this period is the sum of the prevailing temperature and the items in columns 12 and 18. The computed temperature is thus $17.0 + 8.2 + 21.0 = 46.2$ F.

A computation of the effect of daily air temperature changes between day and night indicates that they should normally produce about a 1 F amplitude change under this blanket.

ACKNOWLEDGMENTS

Fig. 9, 10, 11, and 12 were prepared by Q. L. Florey and checked by G. G. Balmer. The experimental data for the 2-ft cube were obtained by L. J. Mitchell.

Addendum to a paper by L. H. Tuthill, R. E. Glover, C. H. Spencer and W. B. Bierce:

Insulation for Protection of New Concrete in Winter*

The following tables of insulation requirements for protection of concrete placed in cold weather were prepared by Q. L. Florey and G. G. Balmer from the charts published in the original paper.

TABLE A—INSULATION REQUIREMENTS FOR CONCRETE WALLS
Concrete placed at 50 F

Wall thickness, ft	Minimum air temperature allowable for these thicknesses of commercial blanket or bat insulation, degrees F			
	0.5 in.	1.0 in.	1.5 in.	2.0 in.
Cement content—300 lb per cu yd				
0.5	47	41	33	28
1.0	41	29	17	5
1.5	35	19	0	−17
2.0	34	14	− 9	−29
3.0	31	8	−15	−35
4.0	30	6	−18	−39
5.0	30	5	−21	−43
Cement content—400 lb per cu yd				
0.5	46	38	28	21
1.0	38	22	6	−11
1.5	31	8	−16	−39
2.0	28	2	−26	−53
3.0	25	− 6	−36	
4.0	23	− 8	−41	
5.0	23	−10	−45	
Cement content—500 lb per cu yd				
0.5	45	35	22	14
1.0	35	15	− 5	−26
1.5	27	− 3	−33	−65
2.0	23	−10	−50	
3.0	18	−20		
4.0	17	−23		
5.0	16	−25		
Cement content—600 lb per cu yd				
0.5	44	32	16	6
1.0	32	8	−16	−41
1.5	21	−14	−50	−89
2.0	18	−22		
3.0	12	−34		
4.0	11	−38		
5.0	10	−40		

*ACI JOURNAL, Nov. 1951, *Proc.* V. 48, p. 253. Disc. 48-18 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

TABLE B—INSULATION REQUIREMENTS FOR CONCRETE SLABS AND CANAL LININGS PLACED ON THE GROUND

Concrete at 50 F placed on ground at 35 F; no ground temperature gradient assumed

Slab thickness, ft	Minimum air temperature allowable for these thicknesses of commercial blanket or bat insulation, degrees F			
	0.5 in.	1.0 in.	1.5 in.	2.0 in.
Cement content—300 lb per cu yd				
0.333	*	*	*	*
0.667	*	*	*	*
1.0	47	42	35	29
1.5	37	19	− 1	−21
2.0	26	− 5	−37	−70
2.5	16	−27	−72	
3.0	6	−51		
Cement content—400 lb per cu yd				
0.333	*	*	*	*
0.667	50	49	47	46
1.0	42	30	17	5
1.5	29	1	−27	−56
2.0	16	−28	−72	−117
2.5	3	−58		
3.0	−10	−86		
Cement content—500 lb per cu yd				
0.333	*	*	*	*
0.667	47	42	35	30
1.0	37	19	0	−19
1.5	21	−16	−54	−92
2.0	5	−51		
2.5	−13			
3.0	−26			
Cement content—600 lb per cu yd				
0.333	*	*	*	*
0.667	43	34	24	14
1.0	31	7	−18	−42
1.5	13	−33	−80	−127
2.0	− 5	−74		
2.5	−22			
3.0	−42			

*Due to influence of cold subgrade on these thin slabs, insulation alone will not maintain their temperature at the required 50 F in cold weatner. In such cases the additional heat supply necessary to maintain required temperatures in the concrete must be provided by using higher placing temperatures, preheating of the ground, electric resistance wire under the insulation, or by other means depending on the severity of prevailing weather.

Notes: The tables are calculated for the stated thicknesses of blanket-type insulation with an assumed conductivity of 0.25 Btu per hr per sq ft for a thermal gradient of 1 deg F per in. The values given are for still air conditions and will not be realized where air infiltration due to wind occurs. Close-packed straw under canvas may be considered a loose-fill type if wind is kept out of the straw. The insulating value of a dead-air space greater than about ½ in. thick does not change greatly with increasing thickness. Textbooks or manufacturers' test data should be consulted for more detailed data on insulations. See insulation equivalents, Table D.

TABLE C—INSULATION REQUIREMENTS FOR CONCRETE SLABS AND CANAL LININGS PLACED ON THE GROUND

Concrete at 50 F placed on ground at 40 F; no ground temperature gradient assumed

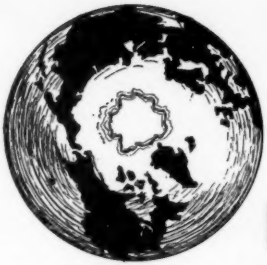
Slab thickness, ft	Minimum air temperature allowable for these thicknesses of commercial blanket or bat insulation, degrees F			
	0.5 in.	1.0 in.	1.5 in.	2.0 in.
Cement content—300 lb per cu yd				
0.333	*	*	*	*
0.667	49	47	44	42
1.0	43	33	22	12
1.5	33	12	−10	−33
2.0	24	− 9	−43	−77
2.5	14	−31	−76	
3.0	5	−52		
Cement content—400 lb per cu yd				
0.333	*	*	*	*
0.667	46	40	32	26
1.0	37	22	5	−12
1.5	25	− 5	−37	−68
2.0	13	−32	−78	
2.5	1	−59		
3.0	−11			
Cement content—500 lb per cu yd				
0.333	*	*	*	*
0.667	42	32	21	10
1.0	32	10	−13	−35
1.5	17	−23	−63	−103
2.0	3	−55		
2.5	−12			
3.0	−27			
Cement content—600 lb per cu yd				
0.333	*	*	48	48
0.667	39	24	9	− 5
1.0	27	− 1	−31	−59
1.5	10	−40	−90	−139
2.0	− 8	−78		
2.5	−25			
3.0	−43			

*See footnote, Table B.

TABLE D—INSULATION EQUIVALENTS†

Insulating material	Equivalent thickness, in.
1 in. of commercial blanket or bat insulation	1.000
1 in. of loose fill insulation of fibrous type	1.000
1 in. of insulating board	0.758
1 in. of sawdust	0.610
1 in. (nominal) of lumber	0.333
1 in. of dead-air space (vertical)	0.234
1 in. of damp sand	0.023

†See notes, Table B.



Letters from Readers*

BY WAY OF SYNOPSIS

A. ZIROS tells about a method of building reinforced concrete bridges without centering.

Test data on preformed expansion joint fillers is presented by LEO KAMPF.

Free Span Construction of Prestressed Concrete Bridges (LR 48-14)

Until recently bridges over deep valleys and broad rivers were usually of steel construction to which alone the free span process, *i.e.*, without scaffolding, could be applied. Concrete, requiring a supporting framework of high initial cost, could not compete. The cases are not rare where the cost of the scaffolding was higher than that of the actual bridge. Beyond a certain span, engineering of the supporting framework was a more complicated problem than was the bridge. Further, bridges long supported by a frame have, one day, when the latter is removed, to rely on their own load-carrying capacity and faulty removal of the scaffolding could endanger the whole structure.

These difficulties are eliminated by the "Spannbeton Dywdag" process conceived by Finsterwald which makes it possible to build concrete bridges without solid scaffolding, *i.e.*, in free span.

This principle was applied for the first time in the construction of a bridge over the Lahn River near Balduistein, Germany.

By this method concreting starts from the banks and is done in work sections of 3 m (9.843 ft). To make concreting possible the frame for a 3 m section hangs on a so-called "free span van" which projects with its front end 3 m into the free space. Placing the frame and fixing the reinforcing steel in place for a 3 m section requires 3 days. Next the concrete is placed. While the concrete hardens—it requires 4 days when high grade cements are used—the working team moves over to the opposite part of the bridge and repeats the framing, reinforcing and concreting of a 3 m section. Thus, week after week a 3 m section is completed on each end which means that work proceeds at the rate of 1 m per working day; a noteworthy accomplishment when compared with time required for construction of large bridges by other methods.

The reinforcing steel is type "Stahl 90" which not only has a uniform diameter of 1.023 in. but also a uniform length of 19.686 ft (6 m). Thus the bars are double the length of a 3 m free span section. One group of bars is anchored in each 3 m section of the bridge formed. The bars are joined together by sleeves as used in normal reinforced concrete construction. Immediately upon hardening of the concrete in one section and upon stressing the bars fixed in it, grout is injected so that for further free span con-

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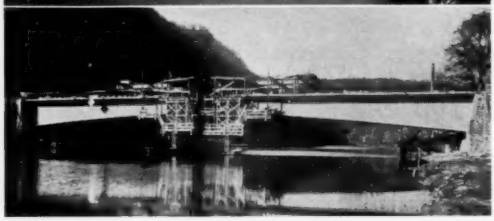
Early stage of free-span construction of prestressed concrete bridge near Balduistein, Germany. Note cantilevered forms and working platforms. Concreting is done alternately from opposite sides.

Nearing completion of bridge. Prestressed sections are self-supporting and carry the weight of the suspended forms as well.



Closing the 205-ft span. After concreting, curing, removal of forms and prestressing of the last concrete section the bridge will be ready for traffic.

struction they are fully bonded to the concrete. The surprising thing is that the reinforcing bars can be stressed as early as 4 days after the concrete has set. Deflection is checked on each 3 m section and can be regulated when necessary.

In the Spannbeton Dywidag process the steel bars are distributed across the tension zone in the same arrangement as in ordinary reinforced concrete construction. The bars are contained in thin-walled sheet metal tubes and are free to move in the concrete until stressed. Stressing is done in a few minutes by easily-handled hydraulic presses weighing less than 40 lb. Bond between concrete and bars is established by injecting grout between the tubes and bars.

The tensioning bars are anchored in the concrete by anchor plates, threads and nuts. The threads on the bars are rolled on cold leaving the threaded zone with the same load-carrying capacity as the remainder of the bar. Friction between the bars and tubes is known so that the friction pressure can be determined mathematically and eliminated by applying an increased pre-loading force.

"Stahl 90" has an ultimate strength of 128,000 psi and an elastic limit of 92,000 psi. As it is an alloy steel which does not owe its strength to cold working it has a high creep boundary (more than 78,000 psi). Thus there are no permanent deformations of the steel within the tensions applied for prestressing.

In addition to the bridge over the Lahn River illustrated here, which was opened to traffic some months ago, another bridge of 236-ft span is under construction over the Neckar near Stuttgart, Germany. Last May the Public Works Dept. of Rheinland-Pfals contracted for the first concrete bridge over the Rhine. This bridge has an over-all length of 1160 ft and a maximum span of 377 ft and is now being built by the free span process.

A. Ziros, director, Dyckerhoff & Widmann, S.A., Pty., Ltd., Johannesburg, S. Africa

Comparative Tests of Preformed Expansion Joint Fillers for Concrete Pavement (LR 48-15)

Tables 1, 2 and 3 which follow are comparative tests of some of the more common preformed joint fillers discussed by Clemmer in the February 1951 JOURNAL.*

The test methods are those described in ASTM D545. The type of filler indicated in column 3 of Table 1 are those referred to in ASTM D544. "Kork Pak" is a cross between Type I and Type V filler and "Elastite" is the bituminous type described in ASTM D994.

LEO KAMPF, chemist - in - charge, Bureau of Engineering, Office of the President of the Borough of Queens, City of New York, N. Y.

TABLE 1—COMPARISON OF JOINT FILLERS

Material	Thickness, in.	Type	Nature of material	Recovery, percent	Extrusion, in.	Penetration of recovered bitumen, 77 F, 100g, 5 sec.	Bitumen content, percent	Comp., psi (Loss after comp.)	Boiling in HCl 1 hour
Cork	½	I	Granulated cork particles securely bound together by an insoluble synthetic resin	90 95[a]	0.079	—	——	490 (—)	OK
Cork	¾	I	Ditto	100	0.03	—	——	275 (—)	OK
Cork (self-expanding)	½	II	Ditto	94.5[a]	0.023[a]	—	——	718 (—)[a]	OK
Cork (self-expanding)	¾	II	Ditto	94[a]	——	—	——	715 (—)[a]	OK
Cork (self-expanding)	1	II	Ditto	97[a]	0.023[a]	—	——	625 (—)	OK
Flexcell[b]	½	V	Cane fiber impregnated with asphalt	75	0.03	64	48.3	281 (less than 1 percent)	—
Kork Pak[c]	½	—	Cork and cottonseed bulbs cemented together with gilsonite and asphalt with a layer of asphalt saturated paper	94	0.08	10	69	565 (less than 1 percent)	—
Kork Pak[c]	¾	—	Ditto	83	0.18	11	70	390 (less than 1 percent)	—
Elastite[d]	½	—	Bituminous fiber with an asphalt saturated paper	72[e]	——	—	——	750 (1 percent)	—
Elastite[d]	1	—	Ditto	—	1.78[f]	16	67	———	—

[a] Specimens air-cured 24 hours only; further tests in Table 3.
[b] Produced by Flexcell Co.
[c] Produced by Servicised Products Co.
[d] Carey Elastite Asphalt Expansion Joint, Philip Carey Mfg. Co.
[e] Specimen compressed 32 percent since the capacity of the machine was reached.
[f] 603 psi needed to compress specimen to one-half its original thickness.
[g] After 2-week period.

*Clemmer, H. F., "Effective Sealing of Concrete Pavement Joints," ACI JOURNAL, Feb. 1951, *Proc.* V. 47, pp. 445-448.

TABLE 2—WEATHERING TEST

Material	Thickness, in.	Type	Freeze-thaw 10 cycles	Recovery, percent	Comp. psi	Water absorption after 10 days, percent
Cork	½	I	OK	—	—	—
Cork	¾	I	OK	97.5	262	34.2
Cork (self-expanding)	½	II	OK	—	—	—
Cork (self-expanding)	¾	II	OK	—	—	—
Cork (self-expanding)	1	II	OK	—	—	—
Flexcell	½	V	OK	78.2	313	88
Kork Pak	½	—	OK	88	690	45.5
Kork Pak	¾	—	OK	91	440	30.2
Elastite	½	—	—	—	—	4.2

TABLE 3—SELF-EXPANDING CORK (TYPE II)

Thickness, in.	Exposure			Expansion, percent	Recovery, percent	Compression, psi	Extrusion, in.
	Hours in air	Hours in H_2O	Temperature H_2O, °F				
½	24	1	212	160.0	—	—	—
	24*	—	—	158.0	90	332	0.029
	—	1	50	101.0	—	—	—
	—	8	50	107.5	—	—	—
	—	24	50	109.5	—	—	—
	—	72	50	113.0	—	—	—
	—	1	70	100.1	—	—	—
	—	8	70	106.1	—	—	—
	—	24	70	123.0	—	—	—
	—	72	70	136.0	—	—	—
	—	3 min.	212	145.0	—	—	—
¾	24	1	212	147.0	—	—	—
	24*	—	—	146.0	87.5	219	0.073
	336†	—	—	128.0	—	—	—
	—	1	50	101.0	—	—	—
	—	8	50	107.5	—	—	—
	—	24	50	109.5	—	—	—
	—	72	50	113.0	—	—	—
	—	1	70	107.2	—	—	—
	—	8	70	118.0	—	—	—
	—	24	70	121.0	—	—	—
	—	72	70	125.2	—	—	—
	—	3 min.	212	140.0	—	—	—
1	—	1	212	166.0	—	—	—
	24*	—	—	164.0	88	163	—
	—	—	—	—	—	—	0.072
	—	1	212	161.0	—	—	—
	24*	—	—	—	—	—	0.079
	48‡	—	—	154.0	92	200	—
	336†	—	—	149.0	—	—	—
	24	—	—	—	—	—	0.023
	—	1	212	163	—	—	—
	24*	—	—	162	88.5	175	—
	24*	—	—	—	—	—	0.092
	24	—	—	—	92	625	—
	—	1	212	168	—	—	—
	24*	—	—	168	—	—	0.27
	24*	—	—	147	92.5	150	—
	336‡	—	—	142	—	—	—

*Air dried an additional 24 hours.
†Air dried an additional 336 hours.
‡Air dried an additional 48 hours.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Shrinkage and creepage of concrete (Ueber das Schwinden und Kriechen des Betons)

E. Friedrich, *Oesterreichische Bauzeitschrift* (Vienna), V. 5, No. 8, pp. 137-141; No. 10, pp. 174-178

Reviewed by Rudolph Fischl

Discusses the problems of deformation of concrete as induced by shrinkage and creepage. The author tries to express this deformation in analytical form, considering the influence of time, climate and other factors. Equations are developed and illustrated by few examples.

The American building industry

The Reinforced Concrete Review (London), V. II, No. 4, Oct. 1950, pp. 247-259

Reviewed by C. P. Siess

These notes, taken from a report by a team from the British building industry that visited the United States in 1949, provide an excellent opportunity for us to see ourselves as others see us. All phases of construction practices are discussed and compared critically, and usually favorably, with British practice.

Torsional stiffness of marginal girders in reinforced concrete floors (Ueber die Berucksichtigung der Torsionssteifigkeit der Randbalken von Stahlbetondecken)

W. Sager, *Beton- u. Stahlbetonbau* (Berlin), V. 45, No. 10, pp. 230-235

Reviewed by Rudolph Fischl

Extensive calculations were previously made by the author to determine the influence of torsion on continuous exterior girders used in building construction. The end conditions of girder, beams and columns, the influence of continuous slabs, the center of rotation and other factors are considered and the results illustrated by graphs.

It is emphasized that the marginal girder has to be regarded as an integral part of the exterior columns and that its dimension should be in accordance with the column dimensions. The author does not recommend the usual practice of keeping the marginal girder small to provide for a brick facing or an expansion joint.

New method for the solution of three-point problems (Un nuovo metodo per la soluzione del problema dei tre punti)

Ugo Fuxa, *Il Cemento* (Milan), V. 47, No. 9, Sept. 1950, pp. 165-166

Reviewed by Gennaro Mianulli

A simple method for the solution of a three-hinged structural system has been worked out. It is a graphical method consisting of a funicular polygon connecting in a given system of forces, the rays passing through assigned forces in three determined points.

Strain gage for embedment in concrete

NBS Technical News Bulletin, V. 35, No. 9, Sept. 1951, pp. 137-139

Reviewed by Rudolph C. Valore, Jr.

Description of an internal strain gage which may be embedded in concrete during the fabrication of a structural member or test specimen. The gage utilizes the A-9 (gage length, 6 in.) or other SR-4 paper or Bakelite gage as the sensing element; it is prepared by using a thermo-setting adhesive

*A part of copyrighted Journal of the American Concrete Institute, V. 23, No. 3, Nov. 1951, *Proceedings* V. 48. Address 18263 W. McNichols Rd., Detroit 19, Mich. Copies of articles or books reviewed are **not** available through ACI. In most cases they can be obtained direct from the original publishers. Address, when available, will be furnished by ACI on request.

to bond the SR-4 element on both sides to the inner surfaces of an envelope made from 0.001-in. or other brass shim stock. The adhesive is cured at room temperature to 160 F. The metal-encased gage, 0.01- to 0.02-in. thick, is embedded in fresh concrete at the desired position and orientation within the specimen or member immediately after placing the concrete. Transfer of strain from concrete to gage element is provided by the bond of the cement-water paste in the concrete to the metal foil after the concrete has hardened. The gage constant determined by the manufacturer of the SR-4 element remains unchanged for the waterproof gage.

Aerated concrete

Civil Engineering and Public Works Review (London), Parts 1 & 2, V. 46, Nos. 540-1, June-July 1951
Reviewed by CHARLES W. DOHN

This digest is concerned with lightweight aerated concrete and not the 5 percent frost resistant type. The structural types are light (35-50 lb per cu ft), moderate (40-65 lb per cu ft) and heavy (70-90 lb per cu ft). Classification of the methods of producing aerated concrete are introduced. General properties of aerated concrete are reviewed. Precast blocks, precast units and *in situ* concrete are considered. Conclusion is made with references to finish, plastering and rendering. Thermal and fireproofing qualities are discussed.

Extreme loading cases for multistory frames (Ungunstige Belastung von Stockwerksrahmen)

R. LAMBERG, *Beton- u. Stahlbetonbau* (Berlin), V. 45, No. 11, pp. 254-256
Reviewed by RUDOLPH FISCHL

The following method is given for computing the maximum and minimum values for the moments in each span:

1. Develop the matrix for rotation of the joints of the system, using the slope-deflexion method.

2. Develop the conjugate matrix, that is the matrix for the rotation of the joints due to a moment $M = 1$ at the individual joints.

3. Compute the joint rotations due to unit moments acting at the ends of each individual beam.

4. With the load members for dead and live load the maximum and minimum moments are calculated for the individual spans.

The results of the calculations are best shown in tables. From the last table it may easily be determined in which spans the live load has to be placed. The total maximum and minimum moments are found by adding the respective values of the partial moments.

An example for a two-story frame with three spans shows that compared with the usual method of total load in all spans the error in the moments at the beams is between 2 and 38 percent and at the columns between 38 and 212 percent.

The author suggests that the conjugate matrix can easily be developed by one of the approximate methods, thus reducing considerably the work involved.

Arch bridge over the Imatra Falls, Finland (Buebro over Imatrafossen, Finland)

C. OSTENFELD and W. JONSON, *Betong* (Stockholm), V. 36, No. 2, 1951, pp. 117-23
AUTHORS' SUMMARY

Short description of road bridge constructed in reinforced concrete in 1950 over the Imatra Falls in eastern Finland to replace a steel bridge more than 50 years old.

The bridge was designed as a flexible arch with overlying stiffening girders, with a 35-m span, similar to several bridges of the famous Swiss engineer, Robert Maillart.

The grouting of the arch was made on formwork suspended under the old bridge, after which the arch took over the role of scaffolding during the construction of the stiffening girders and the superstructure.

Abrasive resistance of pavement increased by the use of vacuum concrete (Execution des dallages au Port du Havre)

I. LEVIANT and E. DE LA SAYETTE, *La Technique Moderne-Construction* (Paris), V. 6, No. 4, Apr. 1951, pp. 160-162
Reviewed by ALEXANDER M. TURITZIN

Reconstruction of the port of Havre posed a problem of choosing a pavement of high abrasive resistance for the heavy-duty loading platforms and warehouse floors. Heavy-duty abrasive traffic is caused by heavily loaded iron-wheeled vehicles operating between storage buildings and loading docks.

For warehouse floors the soil was carefully graded and covered with a layer of sand 10 cm thick. On top of the sand a 12 cm thick slab was placed in bands 2.5 m wide and 7 m long. A vibrated concrete of 300 kg of

cement 250/315 and 180 liters of water per cu m was used. For the loading platform a 13 cm thick vibrated concrete slab was first placed, and later topped with a wearing surface 7 cm thick consisting of a hard aggregate called diorite. As soon as the concrete was placed, it was covered with vacuum concrete suction pans which, it was estimated, removed 27 percent of the mixing water. After the removal of the suction pans, the concrete was finished with wooden floats. At this time, the concrete was so hard that it could carry a man without leaving a surface imprint.

Preliminary tests on concrete slabs showed that the vacuum concrete, in its early days, was at least twice as strong as nontreated concrete permitting its early placement in service. Its final strength was 20 to 25 percent greater, and its resistance to abrasion increased almost threefold. It was observed that removal of the excess mixing water decreased shrinkage and therefore permitted wider spacing of contraction joints.

Use of East Perth Power Station fly-ash as a pozzolan

V. C. Muntand and D. C. Munro, *The Journal of the Institution of Engineers Australia* (Sydney), V. 22, No. 9, Sept. 1950, pp. 207-213

Reviewed by J. R. Shank

Fly ash from the East Perth Power Station in Australia showed admirable qualities for use in concrete. As compared with data published for American fly ashes it was found to be high in silica, high in iron, low in alumina, very low in calcium oxide, low in sulfur and low in carbon content.

Replacements of portland cement by fly ash up to 25 percent showed delays in compressive strength attainment in the early ages but at twelve months all groups containing fly ash replacements had greater strengths than similar specimens containing no replacement in spite of the fact that the higher replacements required more water. The same was found to be true with respect to tensile strength.

Standard lime-mortar test cylinders showed 977 psi at 7 days and 1561 at 28 days.

Cylinders having 25 percent replacement when suspended in sewers noted for quick sulfate action on concrete showed much less deterioration than similar ones without replacement.

Analysis of a reinforced concrete structure with the aid of photoelasticity (Bemessung einer Stahlbetonkonstruktion mit Hilfe der Spannungsoptik)

Rudolf Kuhn, *Der Bauingenieur* (Berlin), V. 26, No. 6, June 1951, pp. 177-181; No. 7, July 1951, pp. 205-207

Reviewed by Aron L. Mirsky

In the Limbach power plant the concrete constituting the roof of the bend in the draft tube also supports part of the scroll and the turbine and generator loads on a span of considerable length (it was not possible to use a center wall in the draft tube for some distance from the section under study). It was necessary that the structure be stiff as well as strong, as any deflection would cause trouble with the machinery.

Fortunately it was possible to treat the section as a simply-supported span with a uniform load symmetrically applied to part of the length. However, in elevation the beam differed markedly from a prismatic beam, so that photoelasticity was resorted to.

The article discusses photoelastic stress analysis and illustrates the results. Of especial interest is Fig. 5, which shows (a) the isoclinics, (b) the isochromatics, (c) the maximum-stress trajectories, and (d) the reinforcement used.

Calculations were made by the shear-difference method and checked, at a section of symmetry, by the slope-equilibrium method. Stresses in the model were then translated into stresses in the prototype structure.

Lightened concrete (Les betons alleges) Part I

J. P. Levy, *Revue des Materiaux* (Paris), No. 423, Dec. 1950, pp. 378-80; No. 424, Jan. 1951, pp. 31-34; No. 426, Mar. 1951, pp. 95-99

Reviewed by Phillip L. Melville

Concretes with a specific gravity lower than 2 are "lightened." They are classified as follows: special gradation and lightweight aggregate concretes; foamed concretes and organic aggregate concretes. The use of lightened concrete has increased especially in Holland, England, Germany and the United States. The use of small quantities of water and cement in no-fines concrete is advocated.

A specific gravity of 1.65 to 1.69 may be obtained with a capillarity of 50 to 75 percent. Thermal conductivity is similar to that of ordinary bricks. The best lightweight aggregates have a structure made of small

voids separated by a vitreous mass. They come from the following sources: natural deposits, such as pumice; manufactured products under various trade names; by-products such as slags and cinders. Finally, the author studies the relation between density and strength for concretes of the first classification. Porosity, resistance to freezing and thawing, thermal acoustical insulation, volumetric changes with temperature, chemical inertia, bond to steel and nailing properties are discussed in some details. Forty references are addended.

Lightened concrete (Les betons alleges) Part II

J. P. Levy, *Revue des Materiaux* (Paris), No. 427, Apr. 1951, pp. 129-132; No. 428, May 1951, p. 165

Reviewed by Phillip L. Melville

The second type of lightweight concrete is obtained by creating a large percentage of small closed voids. Air-entraining concrete does not meet such requirements having an insufficient volume of voids and the voids being too small and opened. Air cells for light weight are obtained by gas producing agents, foaming agents and the use of lime. Common gas producers are: Al, $Zn + CaCl_2$, $H_2O_2 + CaOCl_2$, $C_2Ca + H_2O$. The reaction must be carefully controlled to avoid accidents, toxic or inflammable vapors. Certain gases will react with the reinforcement. Cost is a controlling factor. Strength will vary as a function of density, moisture content, and direction of loading with reference with swelling at time of manufacture.

Pressure of fresh concrete on forms (Druk door vers beton op de bekisting)

A. Guerrin, *Cement* (Amsterdam), No. 7-8, 1951, pp. 127-133

Reviewed by J. W. T. Van Erp

Little attention is usually paid to the design of formwork and this can only be explained by the lack of readily available fundamental design data on the subject. The cost of formwork represents a large enough fraction of the total cost of the concrete job, often more than a few percent, to warrant more time spent on the development of a rational design. Besides, the actual design work should limit itself to only a few fundamental items that would repeat many times throughout the job. A review, therefore, of currently available design know-how on the subject in this country as well as in Europe seems appropriate.

The actual design of formwork is often left to the contractor who is mostly guided by experience and feeling alone. There are many concrete structures of such magnitude that a close design according to exact standards becomes economically desirable. Unfortunately few research data are available so that investigation and analysis, mostly on a theoretical basis, is the only way to develop a complete design method with the aid of these few and scattered data. The problem is much more involved than would seem at first glance.

Some of the conclusions are:

1. The usual assumption of a hydrostatic analogy whereby the pressure of concrete on formwork is simply considered that of a liquid weighing 145 lb per cu ft gives pressures far in excess of the actual ones, and economy of formwork is sacrificed for an oversimplified and easy assumption. This holds particularly true where quick setting cement is used. Furthermore, the tendency to reduce the water-cement ratio and use of stiffer mixes invalidates the analogy even more. The consistency of the mix is of greater influence the smaller the form is. It is of no influence for a concrete mass of infinite extension.

2. The speed of concreting, expressed vertically (in fill-up) in relation to the setting time is important. The pressure increases when the fill-up speed is raised from about 1 ft per hour to 3 ft per hour.

3. The pressure decreases by 25 to 40 percent when the temperature increases from 60 to 100 F.

4. An important factor is the so-called "silo-effect" in tall narrow elements. This effect does not exist in concrete of complete plasticity but is very pronounced in stiff mixes, where the pressure will decrease by 50 percent or more. The effect of vibration, however, offsets this and makes even a stiff mix behave like a fluid during the vibration period.

5. The influence of the angle of friction of concrete against form, whether metal or wood, is little. For instance, the pressure of concrete of fluid plasticity is only 3 percent greater in metal forms ($\delta = 14°$) than in wood forms ($\delta = 23°$).